M/045/002

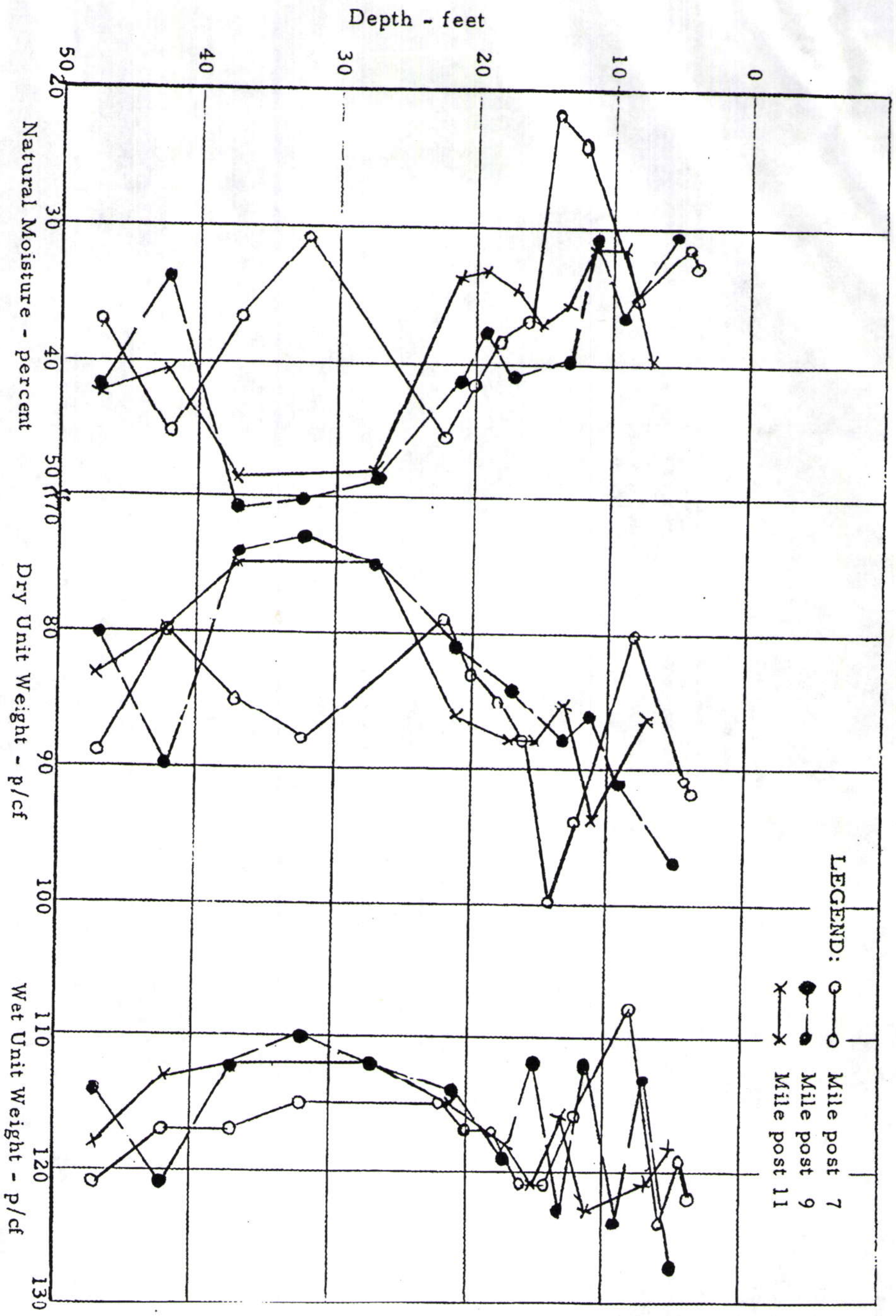


Fig. 6-7. Natural Moisture and Unit Weights

associated with low dry unit weights and vice versa. Fig. 6-7 emphasizes the stratification of the material to a depth of about 20 feet and the uniformity between 20 and 40 feet, with some variation below that depth. Figs. 6-5 and 6-7 both show that there is a great similarity in the materials at mile posts 9 and 11, but that the material at mile post 7 is generally of finer texture. At depths of 32 and 37 feet the material at mile post 7 appears to be of a higher dry unit weight but also of a finer texture than at the other locations. This is an exception to the general observation that the coarser textured soils have the higher dry unit weights.

Consolidation Tests

As mentioned previously consolidation tests were performed on only a few of the clay and silty clay samples to determine some of the consolidation characteristics. The results of these tests are summarized in Tables 6-6 and 6-7. The coefficient of consolidation, Cv, and the primary compression ratio, r, for the various loadings is given in Table 6-6. The void ratio, e, for the various loadings and the compression index, Cc, are given in Table 6-7.

Conclusions

1. The upper 20 feet of the soil material under the salt island is very stratified with layers of oolitic sand of various thicknesses.

2. Throughout the entire depth (approximately 50 feet), the clay material has a very high sensitivity, usually above 10. This was determined by unconfined compression and remolded unconfined samples.

3. Water plasticity ratios of 100 percent or higher indicate that the material would have very little strength when remolded. Therefore, the material is very hard to work with in its present condition.

4. Drying the material seems impractical because the hydrophilic nature of the salt causes the material to reach an equilibrium point at about 28 to 30 percent moisture content.

5. The material may be used as fill material if it can be compacted to reasonable densities and if sufficient granular material is placed above it to distribute the loads. This, however, may be a problem as compaction equipment may not be able to move over it.

Table 6-6. Consolidation Tests --- Coefficient of Consolidation and Primary Compression Ratio

Mile Post	Depth	Coefficient of Consolidation, Cv — Pressure - kg/cm²						Primary Compression Ratio, r — Pressure - kg/cm²					
		0.40	0.79	1.59	3.18	6.36	12.71	0.40	0.79	1.59	3.18	6.36	12.71
	feet	10-4 in²/sec											
7	16	10.0	1.3	1.6	2.0	2.1	2.9	0.49	0.84	0.81	0.60	0.60	0.70
	37	31.8	34.4	23.5	16.6	15.5	4.6	0.26	0.30	0.57	0.22	0.65	0.63
	47	20.2	41.5	26.1	15.2	15.2	10.5	0.22	0.48	0.48	0.40	0.38	0.64
	Average	20.7	25.7	17.1	14.9	10.9	6.0	0.32	0.54	0.62	0.41	0.54	0.66
9	9	5.1	7.0	6.4	9.7	6.2	7.8	0.39	0.46	0.56	0.46	0.69	0.65
	11	----	19.3	26.9	25.5	11.7	4.0	---	0.32	0.44	0.42	0.47	0.44
	15	7.8	9.2	19.1	11.3	10.7	10.9	0.71	0.39	0.39	0.50	0.62	0.72
	19	7.9	13.3	11.4	7.5	4.2	4.7	0.82	0.39	0.14	0.45	0.44	0.68
	21	5.9	13.1	14.5	13.0	3.6	5.0	0.85	0.43	0.45	0.40	0.69	0.78
	27	13.5	15.0	19.2	3.7	9.3	7.9	0.30	0.54	0.34	0.43	0.45	0.98
	37	7.8	5.1	4.8	1.8	9.2	1.9	0.66	0.46	0.66	0.81	1.03	0.76
	42	5.9	4.5	4.0	2.0	1.4	1.3	0.32	0.47	0.61	0.77	0.85	0.68
	Average	7.7	20.8	13.2	9.3	7.0	5.4	0.58	0.43	0.45	0.53	0.65	0.71
11	5	7.3	14.9	10.8	6.4	8.1	9.8	0.73	0.29	0.52	0.40	0.63	0.72
	9	3.0	4.9	11.1	9.7	5.5	---	0.45	0.30	0.62	0.67	0.76	---
	13	10.8	8.3	8.5	5.0	3.0	---	0.43	0.42	0.86	0.64	0.71	---
	Average	7.0	9.4	10.1	7.0	5.5	9.8	0.54	0.33	0.66	0.57	0.70	0.72

Table 6-7. Consolidation Tests—Void Ratio and Compression Index

Mile Post	Depth	Void Ratio, e — Pressure - kg/cm² 0.40	0.79	1.59	3.18	6.36	12.71	Compression Index C_c
	feet							
7	16	0.87	0.83	0.77	0.70	0.60	0.52	0.29
	37	0.46	0.45	0.43	0.40	0.33	0.24	0.31
	47	0.84	0.81	0.79	0.76	0.72	0.68	0.28
Average		0.72	0.70	0.66	0.62	0.55	0.48	0.29
9	9	0.36	0.35	0.34	0.31	0.26	0.17	0.26
	11	--	0.81	0.79	0.75	0.68	0.61	0.26
	15	1.00	0.98	0.94	0.87	0.79	0.71	0.26
	19	0.95	0.93	0.90	0.85	0.80	0.69	0.22
	21	0.97	0.95	0.92	0.87	0.78	0.64	0.33
	27	1.27	1.27	1.21	1.15	0.89	0.59	--
	37	1.23	1.17	1.12	0.99	0.78	0.60	0.62
Average		0.96	0.92	0.89	0.87	0.71	0.57	0.32
11	5	0.85	0.82	0.76	0.69	0.63	0.55	0.27
	9	0.87	0.85	0.83	0.78	0.70	--	0.26
	13	0.95	0.93	0.89	0.83	0.75	--	0.31
Average		0.89	0.86	0.83	0.77	0.69	0.55	0.28
7, 9, 11 Avg.		0.86	0.83	0.79	0.75	0.65	0.55	0.30

6. Sandy material may also be transported and mixed with the onsite material for the lower three to four feet of the fill embankment.

7. With the data available, settlement analysis by the conventional consolidation theory is impractical considering the vertical dessication cracks and the stratification in the upper 20 feet of soil material.

8. The material, however, seems capable of carrying the 5 foot anticipated fill with a probable settlement of less than 6 inches.

9. The material in all other respects is suitable for the lower parts of the embankment.

Part VII

MECHANICAL AND CHEMICAL ANALYSES OF SOIL SAMPLES FROM UNDER THE SALT ISLAND

by

J. E. Christiansen

J. P. Thorne

Introduction

This report includes a presentation and discussion of the mechanical and chemical analyses of soil samples taken by the Utah State Department of Highways at three locations along the line of the proposed Interstate Highway 80 at mile posts 7, 9, and 11. The samples were taken to a depth of about 47 feet and were delivered to the Utah State University Engineering Materials Laboratory in sealed 2-inch diameter Shelby tubes. They were removed from the tubes at the Materials Laboratory, where part of each sample was used for the physical tests reported in Part VI of the Salt Flats Investigations. Portions of 21 selected samples were delivered to the USU Cooperative Soils and Water Laboratory where the analyses reported here were made.

This report has been prepared by J. E. Christiansen from a letter report and data submitted June 7, 1961, by J. P. Thorne, under whose supervision the tests were made. The analyses cover essentially the same items as those reported in Part IV, November 1960, which covered tests on samples from depths of 1.3 to 4.0 feet under the edge of the pavement along the present highway 40 between mile posts 6 and 20. The present samples, therefore, represent a much greater depth of materials, but they are restricted to an area where the hard salt crust varies from about 1 to 3 feet in depth. The report is arranged to facilitate comparisons of the results for the three locations. Some comparisons are also made with results reported in Part IV.

This report follows essentially the same general outline as the previous report, Part IV, except that some of the descriptive matter and explanations are omitted to avoid excessive duplication.

Particle Size Distribution

The soil used for the mechanical analyses for particle size distribution was first treated with hydrogen peroxide to remove organic matter and then washed three times with distilled water to remove the salt. Good dispersion was obtained. This treatment differed from that described in the Report, Part IV. The results of these analyses are summarized in Table 7-1. Eight size fractions, including five subdivisions of sand and two of clay, are given, together with the textural classification which is abbreviated as follows: silty clay, sic; silt, si; sand, s; loam, l; and clay, c.

Table 7-1. Particle Size Distribution

Classification	Particle Size millimeters	Distribution percent of total						
Lab No		U61102	103	104	105	106	107	108
Mile Post 7. Depth, ft		4	8	12	18	22	37	47
Very coarse sand	2.0-1.0	0	1.3	8.0	3.0	1.0	0	0.1
Coarse sand	1.0-0.5	0.2	1.6	12.5	3.6	1.1	0.1	0.2
Medium sand	0.5-0.25	0.2	0.9	18.5	3.4	1.2	0.2	0.2
Fine sand	0.25-0.10	1.4	4.8	45.3	5.4	1.8	1.3	0.8
Very fine sand	0.10-0.05	0.9	2.5	6.9	1.5	0.6	0.8	0.6
Silt	0.05-0.002	41.3	44.0	3.6	35.8	40.6	44.4	44.8
Coarse clay	0.001	28.0	22.4	4.8	20.0	19.9	18.7	25.5
Fine clay	0.001	28.0	22.5	0.4	27.3	33.8	34.5	27.8
Textural classification		sic	sic	s	c	sic	sic	sic
Lab No		U61109	110	111	112	113	114	115
Mile Post 9. Depth, ft		5	9	13	17	21	32	42
Very coarse sand	2.0-1.0	2.0	0.1	0.1	2.3	0.4	0.1	0
Coarse sand	1.0-0.5	8.6	0.2	0.4	5.1	0.7	0.1	0.2
Medium sand	0.5-0.25	9.9	0.3	1.8	4.7	0.9	0	0.1
Fine sand	0.25-0.10	19.8	0.9	13.1	10.1	3.9	·0.1	0.4
Very fine sand	0.10-0.05	7.5	0.8	4.9	8.1	1.7	0.1	0.9
Silt	0.05-0.002	43.2	44.0	35.8	42.8	43.8	37.8	39.8
Coarse clay	0.002-0.001	7.3	24.1	21.7	16.4	24.5	27.1	23.9
Fine clay	0.001	1.7	29.6	22.2	10.5	24.1	34.7	34.7
Textural classification		1	sic	c	1	sic	c	c

Table 7-1. Particle Size Distribution (Continued)

Classification	Particle Size	Distribution						
	millimeters	percent of total						
Lab No		U61116	117	118	119	120	121	122
Mile Post 11. Depth, ft		7	11	15	19	27	37	47
Very coarse	2.0-1.0	0	0.2	0.1	0	0.1	0.1	0.1
Coarse sand	1.0-0.5	0	0.6	0.2	0	0.1	0.1	0.2
Medium sand	0.5-0.25	0	0.8	0.2	0	0.2	0.1	0.2
Fine sand	0.25-0.10	0.3	7.3	1.1	0	0.3	0.4	0.7
Very fine sand	0.10-0.05	0.3	7.4	1.0	0	0.1	0.5	1.4
Silt	0.05-0.002	52.7	25.5	42.6	42.8	41.1	36.8	40.8
Coarse clay	0.002-0.001	13.3	27.9	27.4	34.8	30.7	26.7	23.5
Fine clay	0.001	33.4	30.3	27.4	22.4	27.4	35.3	33.1
Textural classification		sic	c	sic	sic	sic	c	sic

Only one sample, no. 104, from a 12-foot depth at mile post 7, contained an appreciable amount of medium and coarse sand. Six other samples contained more than 10 percent and are classified as sand. Twelve of the 21 samples containing more than 40 percent silt and 40 percent clay are classified as silty clay. The two samples high in silt (more than 40 percent) but with less than 40 percent clay are classified as loams. The six samples with less than 40 percent silt but more than 40 percent clay are classified as clay. Actually there is not a great difference in the samples classified as loams, silty clays, and clays. In all but two instances, the clay fraction is divided quite evenly between coarse clay (greater than one micron) and fine clay (less than one micron). These two exceptions contain relatively more of the coarse fraction.

There is probably no significant difference in the soil texture at these three locations. The log of the borings at the three sites, as shown in the Report, Part VI, indicates that the material to a depth of about 20 feet is highly stratified with a number of thin layers of sand. The samples reported here, which represent depths 4 or more feet apart, do not fully reflect this stratification. It is probable that some of the samples contain a mixture of two or more distinctly different strata.

Moisture and Salinity

Data on soil moisture and salinity are given in Table 7-2. Since all samples were taken from below the water table, the natural moisture percentage does not reflect the textural classification very well. It will be noted, however, that the sandy sample no. 104, from a depth of 12 feet at mile post 7, has the lowest natural moisture percentage, 24, and the lowest saturation percentage, 26. The saturation percentage is usually a fair index of the texture and it checks fairly well for these samples. There are some exceptions, however; for example, sample no. 119 from mile post 11, has the lowest saturation percentage at that location. This sample contains no sand and has 57 percent clay. The relatively low saturation percentage, 47, may be due to the high lime content, 59.2 percent, as shown in Table 7-3.

Electrical conductance determinations were made on both 1:5 soil-water extracts and saturation extracts. The total dissolved solids, which are mostly salts, were determined gravimetrically from the 1:5 extracts. The corresponding percentage of salt on a dry-weight basis (soil plus salt) was calculated from the extract percentage by multiplying by the dilution factor, 5. The salt percentage on this dry-weight basis varies from 7.9 to 16 percent, being the lowest for the sand

Table 7-2. Soils Moisture and Salinity

Lab No	Depth	Moisture Percentage		Electrical Conductance		Dissolved Salt		Soil Solution
		Natural	Saturation	1:5 Extract	Saturation	1:5 Extract	Soil	
	ft	percent	percent	millimhos/cm. at 25°C		percent	percent	
Samples from Mile Post 7.								
102	4	33.0	60	38.15	187.7	2.72	13.6	41.2
103	8	35.2	54	38.15	187.7	2.71	13.5	38.5
104	12	24.0	26	24.80	187.7	1.59	7.9	33.1
105	18	38.4	55	41.33	194.9	2.81	14.0	36.6
106	22	45.6	60	45.08	194.9	3.25	16.2	35.6
107	37	36.9	66	33.06	163.4	2.26	11.3	30.6
108	47	36.8	66	29.17	140.8	2.08	10.4	28.2
Average		35.7	55	35.68	179.6	2.49	12.4	34.9
Samples from Mile Post 9.								
109	5	30.2	53	30.99	187.7	2.20	11.0	36.4
110	9	36.9	61	38.15	174.7	2.49	12.4	33.7
111	13	39.8	52	30.99	180.9	2.03	10.1	25.5
112	17	41.0	37	38.15	202.7	2.43	12.1	29.6
113	21	41.2	57	33.06	187.7	2.06	10.3	25.0
114	32	50.2	63	41.33	194.9	2.90	14.5	28.9
115	42	33.8	66	38.15	158.3	2.64	13.2	39.0
Average		39.0	55	35.83	183.8	2.39	11.9	30.6

Table 7-2 Continued

Lab No	Depth	Moisture Percentage		Electrical Conductance		Dissolved Salt		Soil Solution
		Natural	Saturation	1:5 Extract	Saturation	1:5 Extract	Soil	
	ft	percent	percent	millimhos/cm. at 25°C		percent	percent	
Samples from Mile Post 11.								
116	7	40.0	56	38.15	187.7	2.50	12.5	31.2
117	11	31.4	58	41.33	180.9	2.58	12.9	41.1
118	15	37.3	61	45.08	187.7	3.15	15.7	42.2
119	19	33.3	47	41.33	187.7	2.69	13.4	40.3
120	27	48.2	59	41.33	180.9	2.80	14.0	29.1
121	37	48.7	69	41.33	163.4	2.65	13.2	27.2
122	47	42.0	65	35.42	153.4	2.38	11.9	28.4
Average		40.1	59	40.57	181.9	2.67	13.4	33.3

Natural moisture percentage determined at the Engineering Materials Laboratory.

Total dissolved salts determined gravimetrically from 1:5 extract. Calculated on a dry weight basis (soil plus salt) by multiplying by the dilution, 5.

Table 7-3. Organic Matter, pH, Lime, and Gypsum.

Lab No	Mile Post	Depth	Organic Matter	pH Sat. paste	pH 1:5 Ext.	CaCO3 equiv.	Gypsum	Gypsum
	miles	feet	percent			percent	me./100 gr.	percent
102	7	4	0.88	8.3	9.5	65.7	17.0	1.16
103		8	.77	8.1	9.2	50.4	13.2	0.90
104		12	.74	8.2	9.4	82.7	1.9	0.13
105		18	1.27	7.9	9.0	56.8	29.1	1.98
106		22	.96	7.9	9.0	50.5	22.4	1.52
107		37	.93	7.5	8.8	39.3	<1	0.07
108		47.	.89	7.9	8.8	55.3	39.3	2.68
Average			.92	8.0	9.1	57.2	17.7	1.21
109	9	5	0.58	8.2	8.9	24.1	78.7	5.35
110		9	.89	8.2	9.5	57.8	<1	0.07
111		13	1.00	8.0	9.5	64.7	4.5	0.31
112		17	1.62	7.9	9.5	72.7	6.7	0.46
113		21	.91	8.1	9.4	65.9	<1	0.07
114		32	1.77	7.7	8.9	43.2	6.8	0.46
115		42	1.50	7.6	8.6	44.6	8.6	0.58
Average			1.18	8.0	9.2	53.3	15.3	1.04
116	11	7	0.67	8.5	9.5	58.2	7.8	0.53
117		11	.69	7.9	9.7	59.8	2.5	0.17
118		15	1.12	7.8	9.3	46.0	2.4	0.16
119		19	2.15	7.8	9.1	59.2	7.3	0.50
120		27	1.86	7.6	8.9	47.6	3.6	0.24
121		37.	1.62	7.6	8.7	49.6	1.8	0.12
122		47	1.27	7.6	9.1	35.2	<1	0.07
Average			1.74	7.8	9.2	50.8	3.7	0.25

sample no. 104.

The salt content is also expressed as a percentage of the natural moisture content of the soil. These values range from 25.5 to 42.2 percent, indicating that all, or nearly all, of the salt present, with the exception of gypsum, is in the soil solution rather than crystalized salt. These percentages are approximations only since the natural moisture percentage was determined on a different sub-sample than the salt content.

Chemical Composition of the Salts

The chemical analyses were performed on the 1:5 extracts. The results, expressed in equivalents per million (essentially the same as milliequivalents per liter), are given in Table 7-4. The determinations include all of the principal cations and anions, including carbonate (CO_3) which was not found in any of the samples.

Although there are some differences between the individual samples, the averages indicate no significant differences between the three locations. There is a remarkably good agreement between the total cations and total anions.

Comparing the results at mile posts 7 and 11, it will be noticed that there is, at mile post 7, slightly more sodium salt, 85 percent compared with 81 percent, and more magnesium, 1.9 compared with 1.2 percent. Correspondingly, there is slightly less calcium, 10.3 compared with 11.9 percent, and less potassium, 3.4 compared with 6.0 percent.

Comparing the anions for the same locations, it will be seen that there is less chloride at mile post 7, 90.8 compared with 94.4 percent, and more sulfate, 9.0 compared with 5.4 percent. The bicarbonate accounts for only about 0.2 percent, and there is no difference at the two locations.

The differences in individual samples can be attributed to differences in texture. In general, the coarser textured samples have lower natural moisture contents and less total salt. The amount of salt present at these three locations is approximately the same as found in the roadbed samples at mile post 6, as given in the Report, Part IV, but somewhat

Table 7-4. Chemical Composition of Salt in the Soil Samples on Basis of 1 5 Soil-Water Extracts.

Lab No	Mile Post	Depth	Cations					Anions				Percentages	
			Ca	Mg	Na	K	Total	Cl	SO_4	HCO_3	Total	Na	Cl
	miles	feet	equivalents per million (epm)					equivalents per million (epm)				percent	percent
102	7	4	31.5	9.4	380	14	435	416	26.4	0.65	443	87.4	93.9
103		8	44.3	5.1	388	14	451	419	32.4	.92	452	86.0	92.7
104		12	11.9	7.7	250	11	281	265	7.0	.78	273	89.0	97.1
105		18	63.9	3.6	374	16	458	394	63.6	.69	458	81.7	86.0
106		22	62.2	7.7	456	18	544	494	52.9	.92	548	83.8	90.1
107		37	23.8	4.3	337	16	381	363	17.2	1.20	381	88.5	95.3
108		47	61.3	6.9	270	6	344	289	62.2	.74	352	78.5	82.1
Average			42.7	7.8	351	14	413	377	37.4	.84	415	85.0	90.8
109	9	5	63.0	6.9	282	8	360	297	63.5	0.74	361	78.3	82.3
110		9	43.4	2.6	364	20	430	406	17.0	.74	424	84.7	95.7
111		13	34.9	4.3	291	16	346	326	15.4	.69	342	84.1	95.3
112		17	41.7	3.5	346	18	409	392	16.9	.74	410	84.6	95.6
113		21	30.7	3.4	300	17	351	336	13.7	.83	351	85.5	95.7
114		32	53.7	4.2	442	22	522	464	34.5	.97	499	84.7	93.0
115		42	34.1	17.0	376	18	445	404	33.0	1.11	438	84.5	92.2
Average			43.1	6.0	343	17	409	375	27.7	.83	404	83.9	92.8
116	11	7	68.2	1.7	324	22	416	383	37.4	0.69	421	77.9	91.0
117		11	56.2	3.4	354	28	442	423	12.8	.68	436	80.1	97.0
118		15	64.8	10.2	422	34	531	516	21.6	.83	538	79.5	95.9
119		19	54.5	8.5	368	30	461	437	18.1	1.11	456	79.8	95.8
120		27	55.4	3.4	384	28	471	437	25.1	1.06	463	81.5	94.4
121		37	42.6	3.4	376	24	446	417	27.0	1.06	445	84.3	93.7
122		47	35.8	6.8	331	22	396	368	25.8	.97	395	83.6	93.2
Average			53.9	5.3	366	27	452	426	24.0	.91	451	81.0	94.4

higher than found at the other locations from mile posts 8 to 20. The percentage of sodium is about the same as for the roadbed samples at mile posts 6 and 8 but higher than at the other locations. The percentage of calcium averages less and the potassium more than in the previous samples. The chloride percentages are also higher and the sulfate percentages lower than in the previous samples.

Organic Matter, pH, Lime, and Gypsum

The organic matter content of all samples, as given in Table 7-3, is low but averages nearly twice as high at mile post 11 as at mile post 7, 1.74 percent compared with 0.92 percent. It also averages higher than for the previous samples which were about 0.75 percent.

There are no significant differences in the pH values at the three locations. The values for the 1:5 extracts are higher, 9.2, than for the soil pastes, 8.0, as would be expected because of the depressing effect of the high salt content of the soil pastes. The lime content, $CaCO_3$ equivalent, averages higher at mile post 7, than at mile post 11, 57.2 percent compared with 50.8 percent. These values are slightly lower than for the previous tests. These high lime contents are associated with low base exchange capacities.

The gypsum determinations are highly variable as might be expected because gypsum is often found in relatively large crystals scattered throughout the clay. Individual determinations range from less than one milliequivalent per 100 grams of soil for four samples to 78.7 for the 5-foot depth at mile post 9. Average values range from 17.7 at mile post 7 to 3.7 at mile post 11. These average values are considerably lower than the average values for the previous samples. Gypsum determinations are based on differences in the calcium plus magnesium content determined from a dilute extract, 1:20, 1:50, or 1:100, as compared with a saturation extract where only a small amount of gypsum is in solution. Commenting upon the gypsum determinations, Mr. Thorne had this to say in his letter report:

> Referring again to the gypsum content, there seems no good chemical method of determining this salt. Again the 1:5 extracts indicate the presence of calcium and sulfate where our regular gypsum analysis shows less than one milliequivalent per 100 grams. Our method is based on measurement of the calcium plus magnesium

content of a 1:20, 1:50, or 1:100 solution of the soil, calculation of the calcium plus magnesium found to a soil basis and subtracting from it the amount of calcium plus magnesium on a soil basis found in the saturation extract. The latter is considered to be calcium coming from more soluble salts than gypsum. In soils containing calcium carbonates and high amounts of salt we most likely obtain higher amounts of calcium in the saturation extract than represent the easily soluble calcium compounds since the high salinity has the affect of increasing the solubility of calcium carbonate. Gypsum in pure solution is soluble to the extent of 30.5 milliequivalents per liter. But in most cases both calcium and sulfate exceed this amount in these 1:5 extracts. An analysis for gypsum content based on sulfate content of the two solutions mentioned above rather than calcium plus magnesium may be more realistic in the case of these samples.

Base Exchange Analyses

The base exchange analyses were made by the same procedures described in the Report, Part IV. Determinations were made for the "cation exchange capacity," CEC, the "exchangeable sodium," ESP, and the "exchangeable potassium," EKP. The results are not very satisfactory for the reasons discussed previously. Several of the determinations for exchangeable sodium gave negative values in spite of the large amounts of soluble sodium chloride present in all of the samples.

The cation exchange capacity varies from a low of 4.5 milliequivalents per 100 grams of soil to a high of 18.3. All of the lowest values are for depths of 12 to 19 feet and the highest for depths of 37 and 42 feet. All these values are low because of the high lime content as compared with most soils of this texture. There appears to be a fairly good inverse relationship between the cation exchange capacity and lime content.

The exchangeable potassium percentage was reasonably consistent but somewhat higher than those reported previously, ranging from 19 percent for the sandy sample no. 104 to a high of 85 percent for no. 119. Those reported in Part IV varied only from 13.5 percent to 26.1 percent. The average values are 39 percent at mile post 7, 46 percent at mile post 9, and 62 percent at mile post 11. These are roughly proportional to the soluble potassium percentages.

Those interested in the exchangeable sodium percentage of highly saline soils should read the comments by Mr. Thorne, as quoted on page 16 of the Report, Part IV. Regarding the present determinations, he says:

> The pH values and electrical conductivities of the saturation extracts would classify all of these samples as saline-alkali soils. Other data, of course, show that they are extremely saline. The degree of sodium saturation, however, is a very elusive thing. Perhaps the best evidence of the degree of sodium saturation can be found in the analysis of the 1:5 water extracts which show a high proportion of sodium chloride in all of these samples. Thus, at equilibrium the clay complex must be nearly saturated with sodium. We are unable to explain the results we obtained on some of these samples. In other words, negative values for exchangeable sodium. We do know of certain phenomena that make measurements of exchangeable ions difficult, some of these are explained in the writings of Bower and Reitemeier in the following articles: "Negative Adsorption of Salts by Soils," by C. A. Bower and J. O. Goertzen; "Effect of Moisture Content on the Dissolved and Exchangeable Ions of Soils of Arid Regions," by R. E. Reitemeier. Soil Sci. Soc. of Amer. Proc., Vol. 19, No. 2, April 1955; Soil Sci. 61:195 (1946). In this report, as before, the values given with prime symbol are those obtained on the samples washed with 80 percent alcohol. The EC_e prime values show that washing with 80 percent alcohol was quite effective in removing soluble salts and the amounts remaining in the soil after washing were comparable to those normally found in saline soils. Yet in several cases we were unable to get what appear to be reasonable figures for exchangeable sodium. At the first location the second and third depths gave us negative values. In other words, the water soluble plus the exchangeable sodium). Two negative values were also found at the other two locations. Both measurement of water soluble and total sodium were duplicated in order to see if the negative values were simply due to analytical error. Essentially the same values were obtained upon duplication.
>
> The other extreme also existed in a couple of cases. In other words, the exchangeable sodium seemed to be way too high. One example of this is the upper depth of location No. 2 where the exchangeable sodium percentage is reported as 99 and exchangeable potassium percentage as 38. Exchangeable potassium, on

the other hand, is relatively consistent and potassium forms a relatively high proportion of the exchangeable ions in all of these samples.

Summary and Conclusions

1. The mechanical analyses reported herein, together with the boring logs as reported in Fig. 6-1, (Part VI), page 8, indicate that the soil to a depth of about 20 feet is highly stratified with several layers of sand of various thicknesses interspersed between layers of clay and silty clay. Below 20 feet, the soil is more homogeneous, and all samples were classified as clay or silty clay.

2. The soils are highly saline, containing 8 to 16 percent salt, on a dry-weight basis, of which more than 80 percent is sodium chloride. Nearly all of this salt, with the exception of some gypsum, is in the soil solution at the natural moisture content. Little if any is in the form of salt crystals. Chloride salts predominate with 91 to 94 percent of the total.

3. The organic matter is low, averaging about 1.3 percent. The pH values average about 8.0 for the saturated soil pastes and 9.2 for the 1:5 soil-water extracts. The lime content of the soil is very high, averaging from 51 to 57 percent at the three locations. With some exceptions, lime content is lowest for the samples below 25 feet. The highest percentage, 82.7 percent, is for the sandy sample no. 104. The gypsum content varies widely, ranging from less than one milliequivalent per 100 grams of soil to a high of 79. The average value is about 12, but it ranges from 17.7 at mile post 7 to 3.7 at mile post 11. On a weight basis, the gypsum varies from less than 0.07 to 5.35 percent.

4. Considerable difficulty was experienced in connection with the exchangeable cation determinations. The exchange capacities are low, varying from 4.5 to 18.3 milliequivalents per 100 grams of soil. This low exchange capacity is attributed to the high lime content of the clay. In spite of the high content of sodium salts, some of the determinations for exchangeable sodium gave negative results. Exchangeable potassium percentages range from 19 to 85 percent, averaging about 49 percent.

5. On the basis of these analyses, the soils are classified as highly saline alkali soils.

Part VIII

MOISTURE CONTENT. COMPACTION TESTS AND ATTERBERG LIMITS OF SOIL FROM DRAINAGE CANAL SPOIL BANKS

by

J. E. Christiansen

Dwayne Nielson

J. Derle Thorpe

Harl Judd

Bal Patil

Introduction

A major question that must be answered with respect to the construction of Interstate Highway 80 across the Salt Flats is whether or not it is feasible to use the soil along the proposed route for the highway fill or whether material for the fill must be hauled from selected borrow areas near Wendover and Knolls and, possibly, from the Floating Island located about 12 miles north of the proposed alignment. One of the major costs for this highway project will be the placement of this fill. If suitable material must be hauled from the ends of the 40-mile section between Knolls and Wendover, the maximum haul distance will approximate 24 miles. The nearest suitable material along the highway is at Greyback about 8 miles east of Knolls. If it is found feasible to haul part of this fill from Floating Island this distance may be shortened by a few miles, but to do this will require the construction of a temporary roadway across the salt flats for a distance of about 12 miles. This roadway would have to be of such a type that heavy trucks could travel on it at high speeds. The cost of fill material from any or all of these sources will be high because of the long distances involved.

If, however, the material encountered along the route can be excavated and placed in the fill economically, the possible savings may amount to several million dollars. Whether or not this can be done appears to depend primarily upon the feasibility of placing and compacting this material in the highway fill. Preliminary indications are that, if the material can be properly placed and compacted, much of it may be satisfactory for this purpose. The major question, therefore, is with respect to the placement and compaction of the material, and the principal difficulty is that the material in its present state is much too wet for proper compaction. If this material could be drained and dried to a suitable moisture content before placement and compaction, it would, in all probability, prove to be a satisfactory fill material.

Numberous physical tests have been made on materials taken from depths to nearly 50 feet. These tests indicate that from the surface, or from the bottom of salt crust, to a depth of about 20 feet the material is highly stratified, consisting of layers of silt and clay interspersed with layers of oolitic sand of various thicknesses. Most of the material, however, is classified as silty clay, with some samples classified as clay. The clay fraction generally consists of about 50 percent in the range of one to two microns, and the rest finer than one micron. The soil moisture occupying the pore space is essentially a saturated solution of brine of which about 90 percent is sodium chloride (NaCl). Nearly all of the salt in the soil is in solution rather than crystalized salt, with the exception of gypsum which is found in crystalline form in some of the samples. (See Report, Parts IV and VII.)

Except in very dry years, such as 1961, the depth to the water table over much of the route is within a foot or two of the surface, in many places within inches of the surface. In some areas, however, the water table has been lowered by the excavation of drainage ditches south of the railroad by Bonneville, Ltd., and also by the recent reopening and deepening of the so-called 25-Mile Ditch north of the present highway. The water depth in this drain at mile post 11.6, on August 17, 1961, was about 8 feet; and 300 feet east of the drain the water table was 5 feet deep. Observations indicate that the drains effect the water table for distances of 0.5 to 1.0 mile from a drain. It has been found that the subsoil underlying the salt crust from mile post 7 to the easterly edge of the salt crust at about mile post 12 is highly permeable to lateral movement of water. This condition also exists between mile post 12 and somewhere between posts 15 and 17. The lateral flow takes place in vertical dessication cracks in a blocky clay structure that varies in depth below the surface at different locations from about 3 to 9 feet. (See Report, Part I, II, and X.) Samples taken where the water table is at a depth of about 5 feet indicate, however, that the material is essentially saturated, with a moisture content of 30 to 40 percent.

The questions that need to be answered are: How can this saturated material from near or below the water table be excavated, spread, and compacted in a highway fill? Could it be excavated with a dragline from deep trenches similar to the Bonneville drains, placed in adjacent spoil banks, allowed to drain and dry for a sufficient period of time, subsequently hauled to the fill, and spread and compacted to a desired density?

The thought occurred that this question might possibly be answered in a preliminary way by obtaining soil samples from the different-aged spoil banks of the Bonneville drains for the purpose of determining the natural moisture at different depths from the top of the embankments, together with the optimum moisture content for compaction. To better interpret these comparisons, Atterberg limit tests were also made on the compaction samples. This report presents the results of such tests on samples secured June 15 and 16, 1961.

Obtaining the Samples

Samples were obtained by Derle Thorpe of the State Highway Department, and J. E. Christiansen and Harl Judd of the USU Engineering Experiment Station. Permission to take the samples was secured from Rands Wiley of the Bonneville Potash Co. Mr. Wiley also furnished information relative to the approximate age of the drains and spoil banks. Banks of widely different ages were sampled in order to determine, if possible, the approximate rate of draining and drying of the material when placed in uncompacted embankments along the drains. The material in these embankments is several feet above the water table and would have had opportunity to drain and dry.

The words "drain" and "dry" as used here, have slightly different meanings. Water that "drains" from the soil will also remove some of the salt from the soil, as it has previously been ascertained that most of the salt present in the soil is in the soil solution. Water that is lost from the soil by "drying," or the evaporation of the water, will leave the salt in crystalline form or in a more concentrated soil solution. Tests to determine the salt content of the soil in the spoil banks might, therefore, indicate whether the moisture content has been reduced primarily by drainage or drying.

At most of the locations sampled, the soil was removed from a vertical hole at the top of the embankment with a 6-inch post-hole auger. Usually a sample for moisture content was taken from the surface, sometimes at 0.5-foot in depth, again at 1.0 foot, and each foot thereafter to the natural surface of the underlying salt, or soil, as the case might be. Large samples of about 50 pounds were obtained at only one or two depths in each embankment, usually at about mid-depth. These samples consisted of all material for a depth of 1.0 to 1.5 feet, rather than a mixture of all the material from the surface to the bottom of the hole. Since the embankments represent a mixture of the salt and soil encountered from the surface to the depth of the drain there was no way of determining from what depth any of the material encountered had originated.

Sample Locations and Descriptions

A total of 54 moisture samples and 10 large compaction samples were obtained from 9 different locations. The approximate location and description of these sample sites is given in Table 8-1. A sketch map of the area showing the approximate locations of sites 4 to 9 is shown in Fig. 8-1.

Moisture Samples

The moisture samples were taken directly from the tip of the auger, placed in tinned sample cans, and sealed with plastic tape to prevent drying prior to weighing. These samples consisted of 100 to 246 grams of dry soil. They were weighed on an electronic scale, with the readings taken to the nearest 0.01 gram, then dried in an oven at about 105° C for about 48 hours.

To determine the approximate salt content, the entire sample of oven-dried soil was placed in a jar containing 5 milliliters of distilled water per gram of dry soil, making a 1:5 soil-water extract. The dry soil slaked rapidly. The suspension was shaken several times then allowed to stand until the liquid above the soil was essentially clear. This extract was then diluted by adding 90 grams of distilled water to 10 grams of the extract, making a diluted 1:50 soil-water extract. The conductance was then determined with two laboratory Solubridge conductance meters, models RD-26 and RD-15. These conductance meters were subsequently calibrated by determining the conductance of a sodium chloride (NaCl) solution made up of 10 grams of salt to 1 liter of distilled water. After determining the conductance reading of this solution with both meters, the solution was repeatedly diluted by adding equal volumes of distilled water. These conductance readings for the different dilutions were plotted on two-cycle logarithmic paper against the salt concentration, and the estimated salt concentration of the extracts were read directly from these calibration curves. The average of the determinations for the two meters was used for the final estimate of the salt content which is expressed as a percentage of the dry weight of the soil and as a percentage of the natural moisture content of the sample. This latter figure indicates whether or not the salt in the sample is in solution or if it exceeds the solubility of the salt in crystalline form. The results of these analyses are given in Table 8-2.

Table 8-1. Approximate Location and Description of Sample Sites

Site No.	Section	Township-Range	Height of Embankment ft	Depth to Water in Drain[1] ft	Approximate Age yrs	General Description
1	30	2 S - 18 W	9	6[1]	1	Canal bank, E side of evaporation pond. About 10 mi S of Highway 40.
2	SW cor 35	1 S - 19 W	15	4 on N	2	Embankment between two drains, E of electric pump, 4 mi S of Highway 40.
3	SW cor 35	1 S - 19 W	10	8	15	About 150 ft S of Site 2.
4	W 1/4 cor 18	1 S - 17 W	5.5	7	0.5	E of 25-Mi Drain, 100 ft S of S Pole Line. 1000 ft N of Highway 40. Mile Post 11.6
5	E 1/4 cor 18	1 S - 18 W	6	5	0.3	N of Utah Salt Plant. Embankment hard and salty. Original salt crust 1 ft thick.
6	SW cor 9	1 S - 18 W	5.5	6	0.3	1.7 mi NE of Site 5, 50 ft N of N Pole Line.
7	NW cor 10	1 S - 18 W	5.5	6	0.3	1.1 mi NE of Site 6.
8	SE 1/4 cor 1	1 S - 18 W	5.5	7.3	0.4	1.1 mi N of N Pole Line, 2.1 mi N of Site 4.
9	SE cor 18	1 S - 18 W	4.5	4	18	150 ft S of RR, N of Utah Salt Plant.

[1] Depth to water in drain from original surface of salt crust or soil

[2] Water level in canal about three feet above natural soil surface

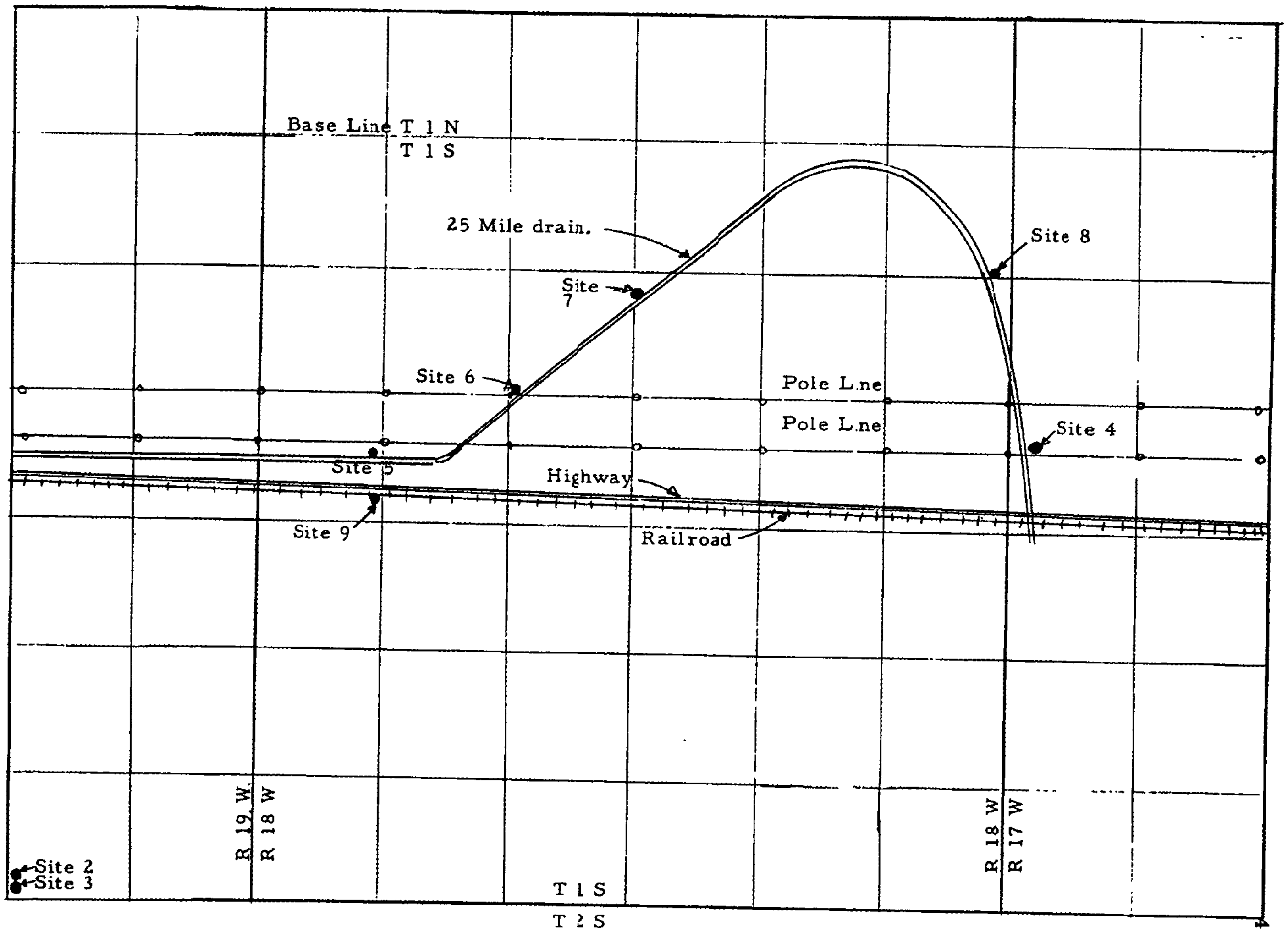


Figure 8-1. Location of sites from which soil samples were obtained from drainage canal spoil banks.

Table 8-2. Moisture and Salt Content of Soil Samples.

Site No.	Can No.	Depth	Natural Moisture	Conductance RB-26	Conductance RB-15	Salt Content Soil Basis	Salt Content Moisture B
		feet	percent	millimhos/cm 25oC		percent	percent
1	19	0.5	17.8	7.60	7.00	24.75	139.0
	20	1.0	36.7	4.60	3.70	12.15	35.8
	21	2.0	35.5	4.20	3.40	12.18	34.3
	22	3.0	24.9	2.55	2.00	6.92	27.8
	23	4.0	38.9	3.45	2.70	9.65	24.8
	24	5.0	35.4	2.20	1.75	5.92	16.7
	25	6.5	30.8	2.70	2.20	7.50	24.3
	26	7.0	31.1	3.55	2.85	10.00	32.1
2	27	1.0	40.0	3.20	2.50	8.92	22.3
	29	2.0	38.2	4.80	3.95	14.25	37.3
	30	3.0	27.9	3.85	3.10	11.00	39.4
	31	3.7	38.6	3.85	3.05	10.92	28.3
	32	4.5	37.6	4.00	3.10	11.25	29.9
	34	5.0	35.8	4.90	3.95	14.25	39.8
	28	7.5[1]	33.8	2.60	2.00	7.00	20.7
3	33	0	6.3	11.60	9.50	37.80	600.0
	35	1.0	28.1	4.05	3.35	11.92	42.4
	36	3.0	29.7	3.60	2.85	10.12	34.1
	1	6.0	25.5	3.65	3.00	10.42	41.1
4	2	0	23.7	1.95	1.50	5.12	21.6
	3	0.5	25.9	2.35	1.78	6.25	24.1
	4	1.0	30.6	3.75	2.95	10.62	34.7
	5	2.0	31.4	4.40	3.50	12.62	40.2
	6	3.0	29.5	3.50	2.90	10.12	34.3
	7	4.0	31.8	3.98	3.10	11.20	35.2
	8	5.0	32.6	3.37	2.70	9.50	29.1
	9	5.5	33.7	3.05	2.30	8.32	24.7
5	10	0	24.0	6.00	4.80	17.00	70.8
	11	1.0	26.9	1.80	1.42	4.78	17.8
	12	2.0	34.0	3.20	2.43	8.67	25.5
	13	3.0	33.6	2.70	2.10	7.32	21.8
	14	4.0	33.5	3.85	2.90	10.62	31.7

[1]Sample taken at edge of water in drain to south of site 2, about 7.5 feet below grour

Table 8-2. Moisture and Salt Content of Soil Samples. (Continued)

Site No.	Can No.	Depth	Natural Moisture	Conductance RB-26	Conductance RB-15	Salt Content Soil Basis	Salt Content Moisture B
		feet	percent	millimhos/cm 25oC		percent	percent
	15	5.0	31.0	2.25	1.75	6.05	19.5
	16	6.0	29.5	2.65	2.00	7.08	30.8
6	17	0	13.9	10.02	8.40	32.12	231.1
	18	1.0	28.8	5.40	4.25	15.62	54.2
	55	2.0	25.5	8.00	6.20	23.80	93.3
	56	3.0	33.5	3.15	2.40	8.58	25.6
	57	4.0	32.8	4.00	3.10	11.25	34.3
	58	5.0	29.5	4.20	3.20	11.70	39.7
7	59	0	22.9	7.40	5.80	22.00	96.1
	60	0.8	28.1	5.30	4.25	15.55	55.3
	61	2.0	18.3	9.40	7.80	29.62	161.9
	62	3.0	31.7	7.40	5.75	21.88	69.0
	63	4.0	33.7	4.50	3.55	12.92	38.3
	64	4.5	33.3	2.55	1.90	6.72	20.8
8	65	0	14.0	10.30	8.40	32.48	232.0
	66	1.0	39.4	5.20	4.20	15.12	38.4
	67	2.0	26.4	7.35	5.90	22.18	84.0
	68	3.0	19.3	10.00	8.20	31.80	164.8
	69	4.0	29.1	6.75	5.60	20.50	70.4
9	70	1.0	36.7	3.35	2.60	9.30	25.3
	71	2.5	32.6	3.22	2.42	8.72	26.7
	72	3.8	32.7	2.75	2.18	7.62	23.2

Compaction Tests

Compaction tests were made in the USU Engineering Materials Laboratory under the direction of Dwayne Nielson. Two methods were used: the Standard Proctor method, AASHO designation T99; and the Modified AASHO method, designation T180. Both tests were made according to standard procedures.

The optimum moisture content for the Proctor and Modified AASHO compaction tests, together with the moisture content for the corresponding samples and the plastic and liquid limits, are given in Table 8-3.

Discussion of the Results

Natural Moisture and Salt Content. The natural moisture content of the soil in the embankment, below at depth of 1.0 foot, varied from a minimum of 18.3, to a maximum of 40 percent. Most of the samples at the surface and at 0.5 foot are somewhat lower in moisture content than the average below that depth, indicating that drying does not occur to any appreciable extent below a depth of 1.0 foot.

The salt content, expressed as a percentage of the moisture content, also indicates this same conclusion. When this salt percentage exceeds this solubility limit of the salts, approximately 32 percent, it indicates that some of the salt is present in crystalline form. A part of this may be gypsum that has dissolved in the 1:5 soil-water extracts. Where the percentage figure is high, it indicates that salt has precipitated on drying, as for the surface samples at sites 1, 3, 5, 6, 7, and 8, or that some of the crystalline salt from the salt crust is mixed with the soil in the sample. At no place in the embankments were large hard blocks of salt encountered, but at several places in the younger embankments, sites 4 to 8, granular salt crystals were encountered. Apparently the blocks of salt removed from the hard salt crust are broken up when dropped onto the embankment, or the blocks disintegrate in the embankment. The high salt percentages, on a soil-moisture basis, at depths of 1.0 foot or more, apparently indicate the presence of the coarse granular crystals originating from the salt crust. It will be noticed that

Table 8-3. Compaction and Atterberg Limit Tests

Site No.	Sample No.	Depth	Proctor Method Optimum Moisture Content	Proctor Method Dry Unit Weight	Mod. AASHO Method Optimum Moisture Content	Mod. AASHO Method Dry Unit Weight	Natural Moisture Depth	Natural Moisture Content	Plastic Limit	Liquid Limit	Plastic Index
		feet	percent	lb/cf	percent	lb/cf	feet	percent	percent	percent	percent
1	1-A	2.0-3.0	23	102	16	110	2.0	35.5	20.1	29.0	8.9
							3.0	24.9			
1	1-B	5.0-6.5	20	106	17	115	5.0	35.4	18.6	29.0	10.4
							6.5	30.8			
2	2-A	3.5-5.0	22	104	17	112	3.7	38.6	19.4	31.0	11.6
							4.5	37.6			
							5.0	35.8			
3	3-A	3.0-4.0	21	106	17	113	3.0	29.7	18.7	28.3	9.4
4	4-A	1.5-2.5	19	105	16	112	1.0	30.6	21.3	30.2	8.9
							2.0	31.4			
							3.0	29.5			
5	5-A	2.0-3.7	18	99	16	114	2.0	34.0	21.3	29.7	8.4
							3.0	33.6			
							4.0	33.5			
6	6-A	2.5-4.5	19	105	18	112	2.0	25.5	19.4	27.3	7.9
							3.0	33.5			
							4.0	32.8			
							5.0	29.5			

Table 8-3 (Continued)

Site No.	Sample No.	Depth	Proctor Method: Optimum Moisture Content	Proctor Method: Dry Unit Weight	Mod. AASHO Method: Optimum Moisture Content	Mod. AASHO Method: Dry Unit Weight	Natural Moisture: Depth	Natural Moisture: Content	Plastic Limit	Liquid Limit	Plastic Index
		feet	percent	lb/cf	percent	lb/cf	feet	percent	percent	percent	percent
7	7-A	2.5-4.0	19	105	14	116	2.0	18.3	21.1	27.4	6.3
							3.0	31.7			
							4.0	33.7			
8	8-A	1.5-2.5	20	104	14	112	1.0	39.4	19.8	31.5	11.7
							2.0	26.4			
							3.0	19.3			
9	9-A	1.0-2.5	21	103	18	111	1.0	36.7	22.3	33.8	11.5
							2.5	32.6			

such salt occurs at the 2.0-foot depth at site 6, and at three levels at 7 and 8.

Much of the variation in the natural moisture content below depths of 1.0 foot is believed to be due to the textural variation. This is also indicated by the Atterberg limit tests. There appears to be no significant differences in the moisture content below a 1.0-foot depth of the two old embankments, sites 3 and 9, and the remainder of the sites, several of which were only a few months old. This would lead to the conclusion that the little drainage that takes place, occurs during the first few months. After that, the drainage rate is so slow as to be negligible. The moisture content in these embankments appears to remain appreciably above the optimum moisture content for compaction.

Compaction Studies. The compaction studies indicate that this material can be compacted to a Standard Proctor dry unit weight of about 104 pounds per cubic foot at an average optimum moisture content of about 20 percent, expressed on the dry-weight basis. This optimum moisture content is about 63 percent of the natural moisture content of the corresponding samples. For the Modified AASHO tests, the average dry unit weight was approximately 112 pounds per cubic foot at an optimum moisture content of about 16 percent. This optimum moisture content corresponds to 52 percent of the natural moisture content of the corresponding samples.

If this material could be loaded, hauled to the fill, and spread in thin layers, some additional drying would occur during the placement and compaction. It appears, however, that the material in the embankments, except for a few inches at the surface, is too wet for satisfactory compaction. The rate of drying is very slow, probably because of its low permeability to movement of moisture, its high salt content, and the hygroscopic nature of the salts.

Atterberg Limit Tests. The plastic and liquid limit tests, the plastic index, and the plasticity index all indicate that the material in the spoil banks would be reasonably satisfactory for highway fills if it could be placed and compacted satisfactorily.

Presence of Salt in the Embankments. Indications are that the salt present in the embankments is mostly in the form of a saturated brine and that crystalline salt occurs only near the surface where it has been deposited by moisture evaporation or where it has originated from the salt crust. This latter source of salt could be readily eliminated by separating the salt crust and the underlying subsoil materials during the excavation by piling the salt crust on one side of the trench and the soil on the other, forming alternate rows of salt and soil. A suggested layout for the excavation and the embankments on the hard salt crust is shown in Fig. 8-2.. This arrangement would facilitate the loading of the material and minimize the hauling distance.

The presence of a saturated brine in the soil is believed to present no serious problem with respect to its use for highway fills.

The layout suggested in Fig. 8-2 is schematic only; no attempt has been made to determine the minimum distance from the highway fill for placing the trenches or the optimum size, spacing, and length of the trenches. In the area illustrated, the presence of the thick salt crust would facilitate the excavating and hauling of the materials to the fill. It is assumed that the fill would be placed and compacted on the salt crust. Removal of the salt crust and exposure of the saturated soft clay and silt beneath it would probably present some additional serious problems in fill placement and compaction. Excavation and hauling of soil materials from borrow areas to the east of the salt crust may also present some difficult problems.

Summary and Conclusions

1. The soil in the spoil banks of the drainage ditches remains at a high moisture content for an indefinite period of time. Average moisture percentages for samples taken from a depth of 1.0 foot or more range in moisture content from about 25 to 40 percent, dry-weight basis, depending on the texture and presence of crystalline salt.

2. Compaction studies indicate that the optimum moisture content for compaction is approximately 20 percent, varying somewhat with texture.

3. Soil samples taken from near the surface and from a depth of 0.5 foot indicate some drying and, consequently, an increase in salt content when expressed on a soil-moisture basis.

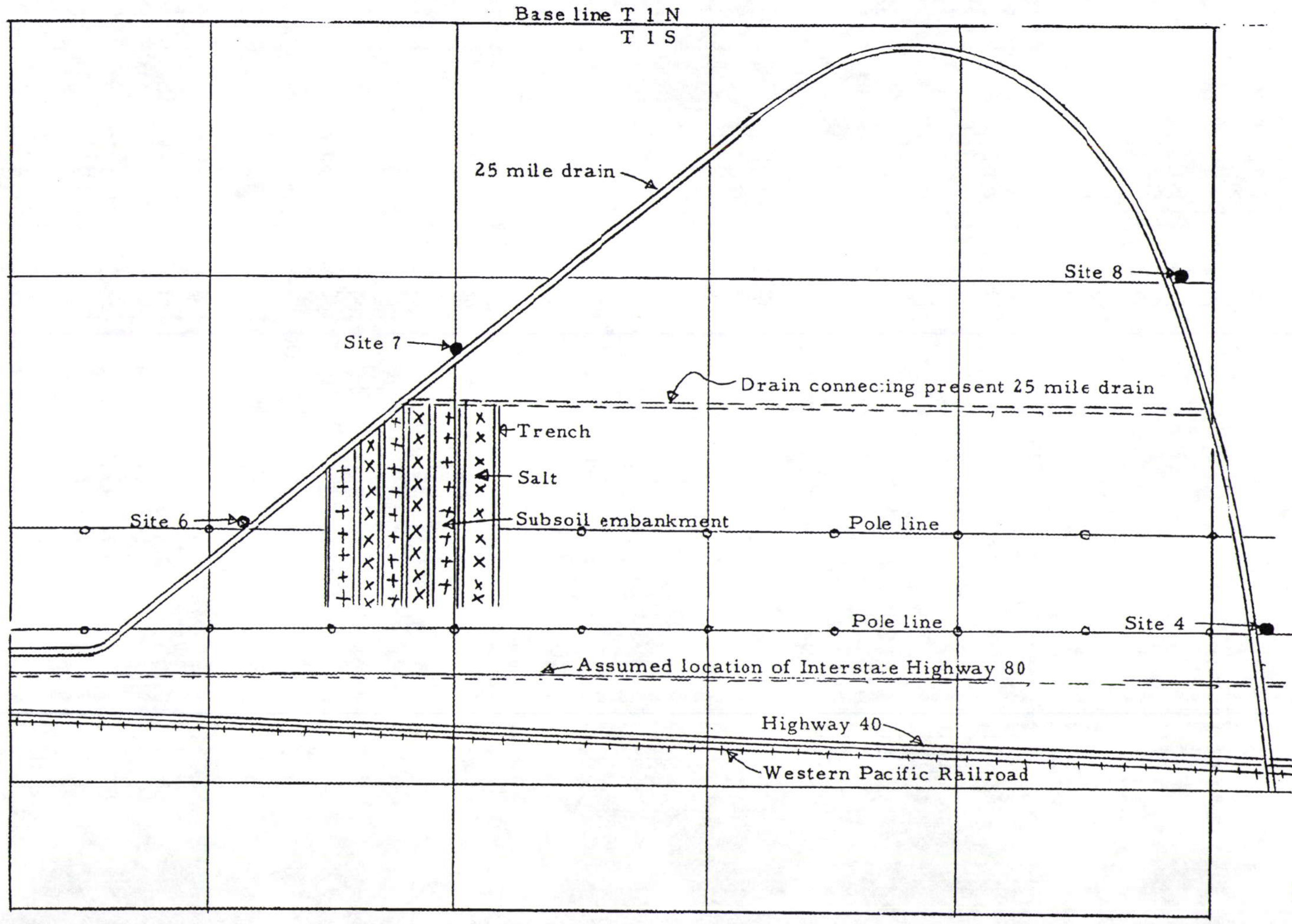


Figure 8-2. Schematic arrangement of trenches and embankments for fill material for Interstate Highway 80 over Salt Flats.

4. Other tests performed as a part of this and other studies indicate that the material would be suitable for fill material if it could be placed and compacted in the fill economically.

The following tentative conclusions appear justified:

1. The material in the embankments is too wet for proper placement in a highway fill.

2. The rate of drying is so slow and the depth to which drying occurs is so shallow that it is not feasible to pre-excavate the material and deposit it in embankments to reduce the moisture content before placing it in a highway fill.

3. Moisture contents in the embankment, as compared with the natural moisture content of undisturbed samples of the subsoil as reported in the Report, Parts VI, and IX, indicate that relatively little moisture is lost from the embankments by drainage.

4. From the available information it would appear that it would be extremely difficult, if not impossible, to excavate material along the proposed route and place it in a highway fill and meet standards for density as required for Interstate Highways.

Recommedations

To more fully determine the problems involved, however, it is recommended that:

1. A short section of an actual highway fill be constructed on the salt crust from material taken from the spoil bank of the 25 mile ditch to determine:
   (a) the construction and material handling problems involved.
   (b) the densities that can be obtained with practical construction techniques.
   (c) whether or not practical and economical techniques can be developed for utilizing the along-route materials.

2. That a second short section be constructed from materials taken directly from beneath the salt crust for the same purposes as enumerated in item 1.

Recommendations (Continued)

3. That during and following the construction of the test embankments specified in items 1 and 2, that a soil moisture and density sampling program be followed to determine the actual moisture contents, densities, and drying rates.

4. That a more detailed program of soil moisture samples be conducted on selected spoil banks to determine the possible changes in soil moisture during the summer drying period.

Part IX

FIELD PLATE BEARING TESTS ON SALT FLATS

by

J. E. Christiansen

Dwayne Nielson

Bal Patil

Introduction

During the week of July 25-28, and again on August 16-17, 1961, plate bearing tests were made at several locations near or along the proposed route of Interstate Highway 80 east of Wendover, Utah. Three of the tests were made west of the salt bed near the Bonneville Ltd. potash plant and the others to the east of the easterly portion of the 25-mile drain.

Purpose

The purpose of these tests were twofold:

1. To work out satisfactory techniques for making such tests using available equipment.

2. To make some preliminary determinations of the bearing capacity of the soils, both at the surface and at various depths below the surface.

Equipment

The basic piece of equipment used for these tests was the California bearing ratio field test equipment furnished by the Utah State Highway Department. This consisted of a special gear-driven jack for applying the load, two proving rings with dial gages for measuring the applied load, one with a 2,000-pound and the other with a 6,000-pound load capacity, together with a penetration dial and auxillary equipment for measuring the penetration of the plate.

A special mounting bracket for mounting the jack on the rear end of a truck and four circular bearing plates of different areas were constructed by the Utah State University Research Foundation. The mounting bracket was bolted to the rear end of a pickup truck owned by Dwayne Nielson. To minimize deflections of the rear end of the truck when the load was applied, two 4x12-inch planks were borrowed from the Research Foundation and used under the rear wheels of the truck and fastened to the rear bumper by means of special clamps constructed for this purpose. This arrangement worked very well for loads up to about 2,000 pounds, at which time the deflection of the planks became excessive.

The bearing plates were made from 5/8-inch steel plate with half of a 1.25-inch pipe coupling welded concentrically to one face. A special adapter was made to connect the 1.5-inch national fine thread of the CBR equipment to the 1.25-inch pipe thread. Several lengths of 1.25-inch galvanized pipe, varying in length from close nipples to 4-foot lengths were provided so that tests could be made at any depth below the surface up to about 13 feet.

A special magnetic mounting for the penetration dial provided a very convenient means of adjusting the dial to a zero reading at the beginning of each test. An 8-foot hardwood beam, 2x6 inches, was used with the dial gage to determine the penetration of the plates into the soil.

The four bearing plates were constructed with the following dimensions:

Diameter inches	Area square inches	Circumference inches
3.571 ± .005	10	11.22
5.051 ± .005	20	15.87
7.143 ± .005	40	22.44
10.102 ± .005	80	31.74

Procedure

Since there is no standardized procedure for making plate bearing tests or for interpreting the results in the field, some time was spent in trying out various techniques in order to arrive at a procedure that would give consistent results, and would not be excessively time consuming.

Tests 1, 2, and 3 were made at mile post 15 with the 10-square-inch plate. Tests 1 and 2 were near the well used for the transmissibility tests, and test 3 was near the north-pole line, about 1/2 mile to the north.

Test 1 was made by turning the crank on the jack at a normal speed until the dial on the 2,000-pound proving ring read 0.0010 inches, for which the load was 39 pounds. This load was then held constant until the end of a 2-minute interval by slowly turning the crank as the penetration increased. At 2-minute intervals the penetration dial was read and the load dial increased by another 0.0010 inches. When the load dial reading of 0.0030 inch was reached, the corresponding load of 116 pounds was held constant while eight consecutive penetration readings were made at 2-minute intervals. This process was repeated, but with only one to three penetration readings at each increment of load until a dial reading of 0.0070 inch was reached corresponding to a load of 271 pounds, or 27.1 psi. This was the maximum load the soil would withstand. This load was held constant while the penetration increased from 0.611 inch to 0.785 inch. This test required a total of 77 minutes, which was considered excessive.

Test 2 differed in that only one penetration reading was made at one-minute intervals until the maximum load reading of 0.0110-inch, or 426 pounds was reached. Again the load was applied and held constant until the end of the one-minute interval when the penetration dial was read. The total time for this test was 10 minutes.

Test 3 was made by the same procedure as test 2, but at a location about 1/2 mile to the north, near the north pole line, on less stable soil. Here the maximum load was only 194 pounds for a penetration of 0.970 inch. The total time required was only 6 minutes.

The results of tests 1, 2, and 3 are shown graphically in Fig. 9-1. The differences between tests 1 and 2 are probably due mostly to the differences in the rate at which the load was applied. The two locations were only about one foot apart.

Tests 4 and 5 were also made near the well at mile post 15, using the 10-square-inch plate for test 4 and the 20-square-inch plate for test 5. For both of these tests the crank on the jack was turned at a slow steady rate. For test 4 both the load and penetration dials were read at 15-second intervals for a total of 6 minutes. For test 5 the procedure was similar, but readings were made at 1/2-minute intervals for a total of 9.5 minutes. The results are shown in Fig. 9-2. The objection to this procedure is that the rate of load application would vary with the resistance to penetration and rate of cranking and could not be duplicated precisely.

For tests 6 and 7 a different technique was tried. The 20-square-inch plate was used for test 6, and the 40-square-inch plate for test 7. The crank was turned at a slow steady rate until the load dial read 0.0010 inch, corresponding to 39 pounds. The crank was then held stationary allowing the load to decrease as the penetration progressed, with readings on both dials being taken at 1/2-minute intervals. When the rate of penetration had dropped to about 0.001 inch per minute, the next load increment was applied and the procedure repeated. After 40 minutes, readings were taken only at 2-minute intervals. A total of 84 minutes was required for the test. Figure 3 shows how the load decreased as the penetration increased for each increment of load. The curve was plotted through the lowest points.

The procedure for test 7 was similar except that readings were taken only at 1-minute intervals for 10 minutes, then at 2-minute intervals for a total of 103 minutes. This procedure was considered too time consuming to be practical. Only two tests were completed during the forenoon of July 26.

For all of the remaining tests, the procedure was essentially as follows: The load was applied at an approximate constant rate of 1.93 psi per minute regardless of plate size. Readings were taken on both dials at 1-minute intervals. At this rate of loading a test could be performed in 25 minutes or less, depending upon whether the load limit of 2,000 pounds, or the penetration limit of 1.0 inch, was reached first. This procedure standardized a slow rate of loading that could be duplicated and permitted a test to be made in a reasonable length of time. Tests 8 to 12 were completed during the afternoon of July 26.

Presentation of Results of Tests

The results of the tests are presented in Table 9-1 and Figs. 9-1 to 9-17. Data on the unit weights of the soil and the moisture content is presented in Table 9-2.

With only two exceptions, Figs. 9-12 and 9-16, all plate bearing data have been plotted to the same scale. The ordinate is the load in pounds per square inch (psi) plotted to a scale of 10 psi per inch, and the abscissa is the penetration of the plate into the soil in inches plotted to a scale of 0.1 inch per inch. In Fig. 9-12 the load is plotted to a scale of 20 psi and in Fig. 9-16 the penetration is plotted to a scale of 0.2 inch per inch.

Table 9-1. Plate bearing tests on Salt Flats

Test No.	Mile Post	Specific Location	Depth	Plate Area	Modulus of Deformation Straight Line	Modulus of Deformation 0.2 in. Penetration	Modulus of Deformation 0.5 in. Penetration	Load At 0.2 in. Penetration	Load At 0.5 in. Penetration
	miles		in.	sq. in.	psi/in.	psi/in.	psi/in.	psi.	psi.
1	15	62 ft N of well	0	10	156	106	20	18.2	25.8
2	15	61 ft N of well	0	10	267	52	23	21.2	32.0
3	15	225 ft N of N pole line	0	10	82	26	12	9.8	15.1
4	15	50 ft N of well	0	10	80	77	34	16.1	31.0
5	15	50 ft N of well	0	20	43	43	23	8.7	20.0
6	15	50 ft N of well	0	20	102	39	21	12.4	20.2
7	15	50 ft N of well	0	40	89	31	20	10.3	17.2
8	15	50 ft N of well	0	40	82	40	22	13.2	24.2
9	15	50 ft N of well	0	20	78	48	26	12.7	24.2
10	15	50 ft N of well	0	80	54	35	17	10.0	17.2
11	15	50 ft N of well	28.5	40	182	96	58	34.2	53.0
12	15	Well at M.P. 15	144	40	35	22	29	7.6	15.1
13	3	1/2 M. N-NW Bonn. Pt.	0	40	105	81	43	20.2	38.0
14	3.5	(1/2 way between pole	0	40	75	57	117	12.1	37.0
15	3.5	(lines N of Bonn. Pt.	3.5	40	216	208	--	42.0	--
16	11.6	(300 ft E of E 25 M. drain,	14	40	60	18	13	8.0	12.3
17	11.6	(Midway between pole lines	0	40	280	272	--	52.5	--
18	12.65	Midway between pole lines	0	40	146	69	21	23.0	34.3
19	12.65		4.5	40	49	49	22	10.0	21.7
20	25	100 ft E NE of well	0	40	115	74	43	19.8	38.0

Table 9-1. Plate bearing tests on Salt Flats (Continued)

Test No.	Mile Post	Specific Location	Depth	Plate Area	Modulus of Deformation			Load At	
					Straight Line	0.2 in. Penetration	0.5 in. Penetration	0.2 in. Penetration	0.5 in. Penetration
	miles		in.	sq. in.	psi/in.	psi/in.	psi/in.	psi.	psi.
21	25	100 ft E NE of well	7.5	40	570	540*	--	53.0*	--
22	25	100 ft E NE of well	37.75	40	199	46	16	24.2	32.0
23	11.6	(100 yd E of drain,	56.5	40	420.0	15.0	10.0	29.2	32.9
24	11.6	(100 ft S of S Pole line	88.0	40	242.0	134.0	--	41.3	--
25	11.6		103.0	40	225.0	84.0	73.0	25.0	47.0
26	11.6	50 ft., S of No., 23	0	10	167.0	142.9	--	20.0	58.0
27	11.6	50 ft., S of No., 23	10.5	40	107.0	21.0	8.0	9.2	13.2
28	11.6	4 ft W of 27	12.25	80	48.7	29.0	10.2	8.2	13.0
29	15	160 ft N of well	0	40	125.0	35.0	23.0	12.5	20.5
30	15	160 ft N of well	0	20	196.7	43.0	30.0	15.0	26.0
31	15	160 ft N of well	0	10	161.2	47.0	34.5	15.2	27.0
32	15	160 ft N of well	0	80	80.7	29.5	20.0	10.0	17.0
33	15	160 ft N of well	40.5	40	203.3	22.2	14.3	17.0	21.8
34	15	160 ft N of well	51.75	40	26.7	16.4	4.5	5.2	7.4
35	15	160 ft N of well	76.0	40	123.7	72.8	19.4	18.7	26.6
36	15	160 ft N of well	94.0	40	166.6	163.0	--	37.8	--

* At 0.1-inch penetration

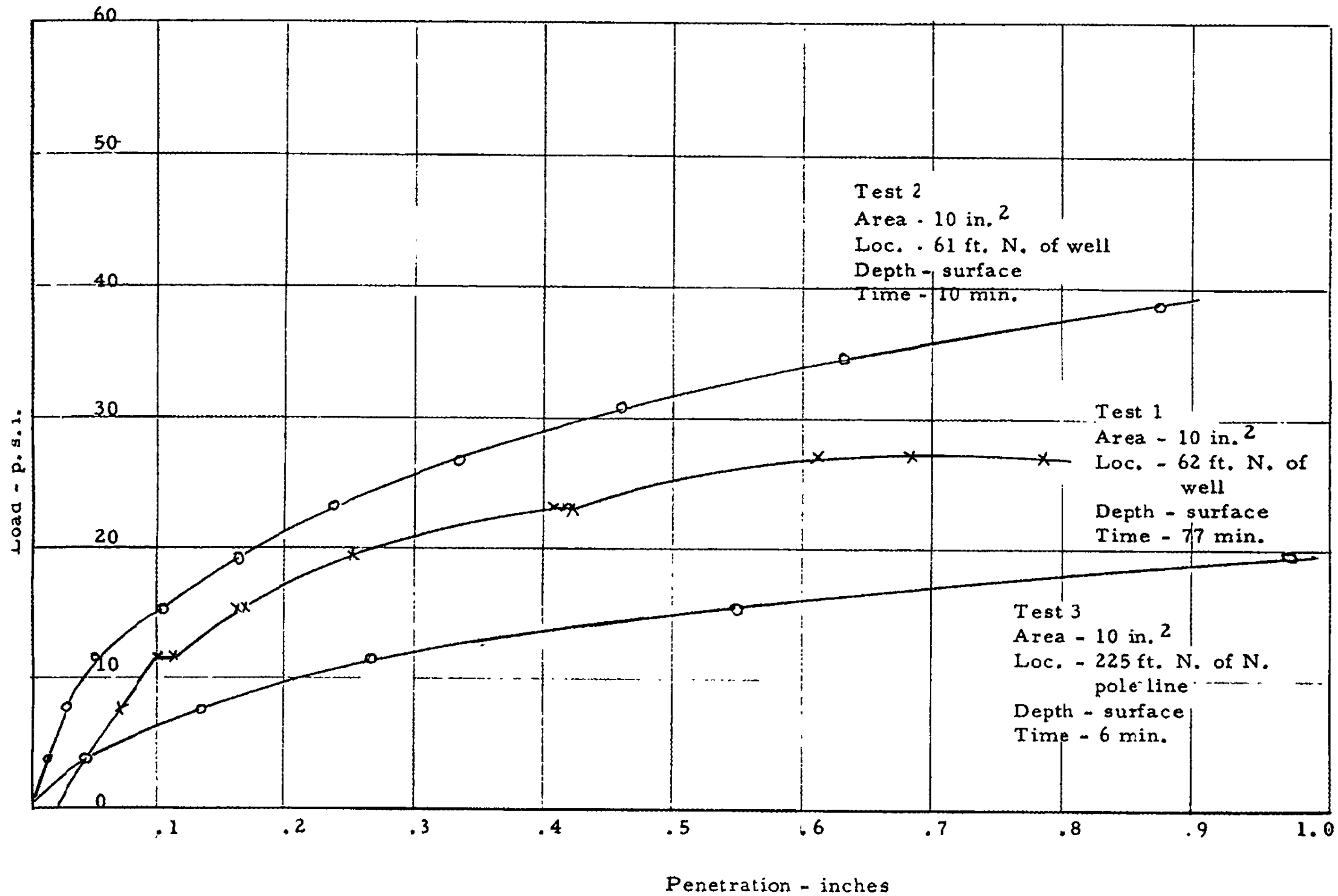


Fig. 9-1. Plate Bearing Tests at Mile Post 15

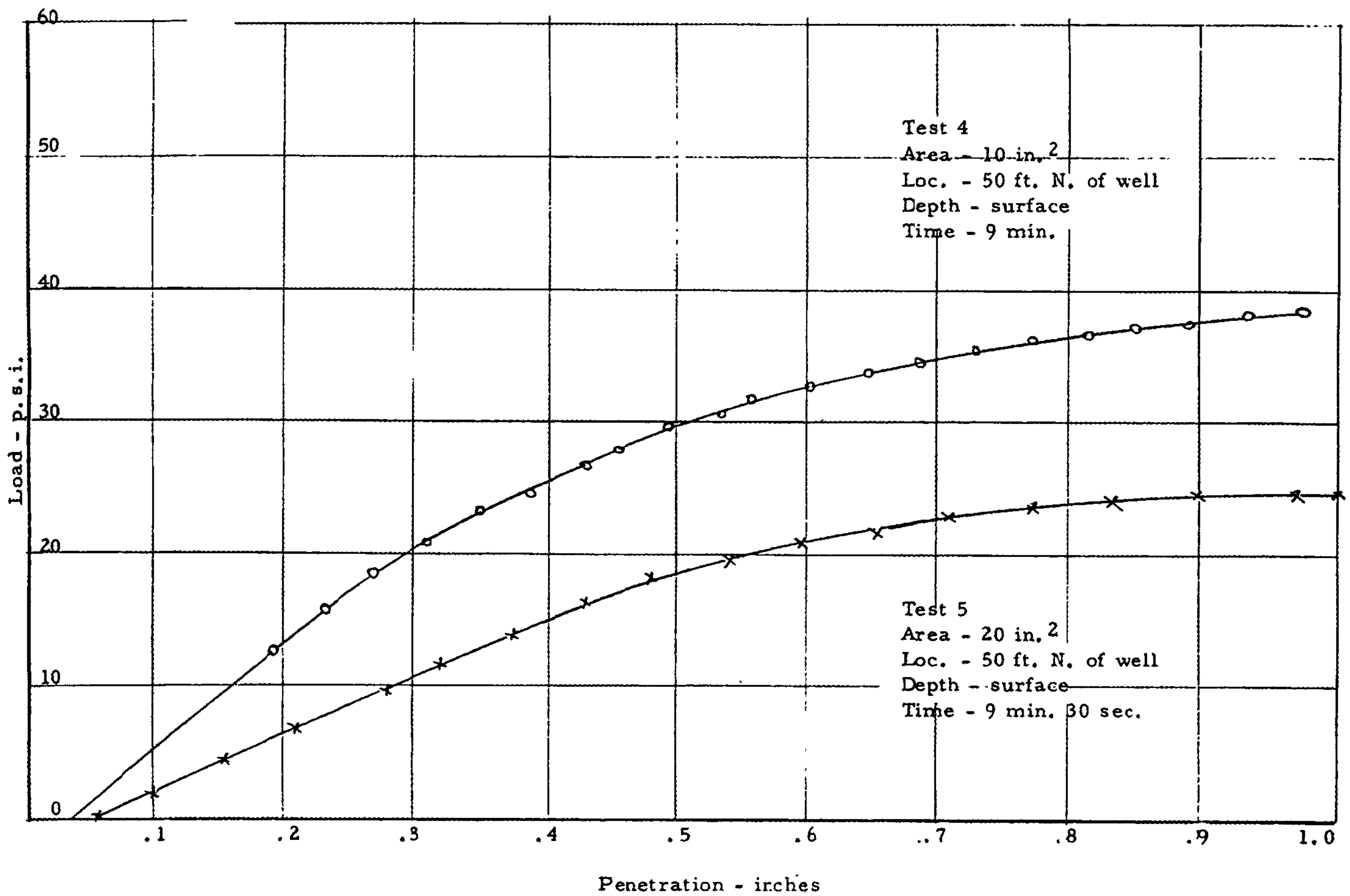


Fig. 9-2. Plate Bearing Tests at Mile Post 15

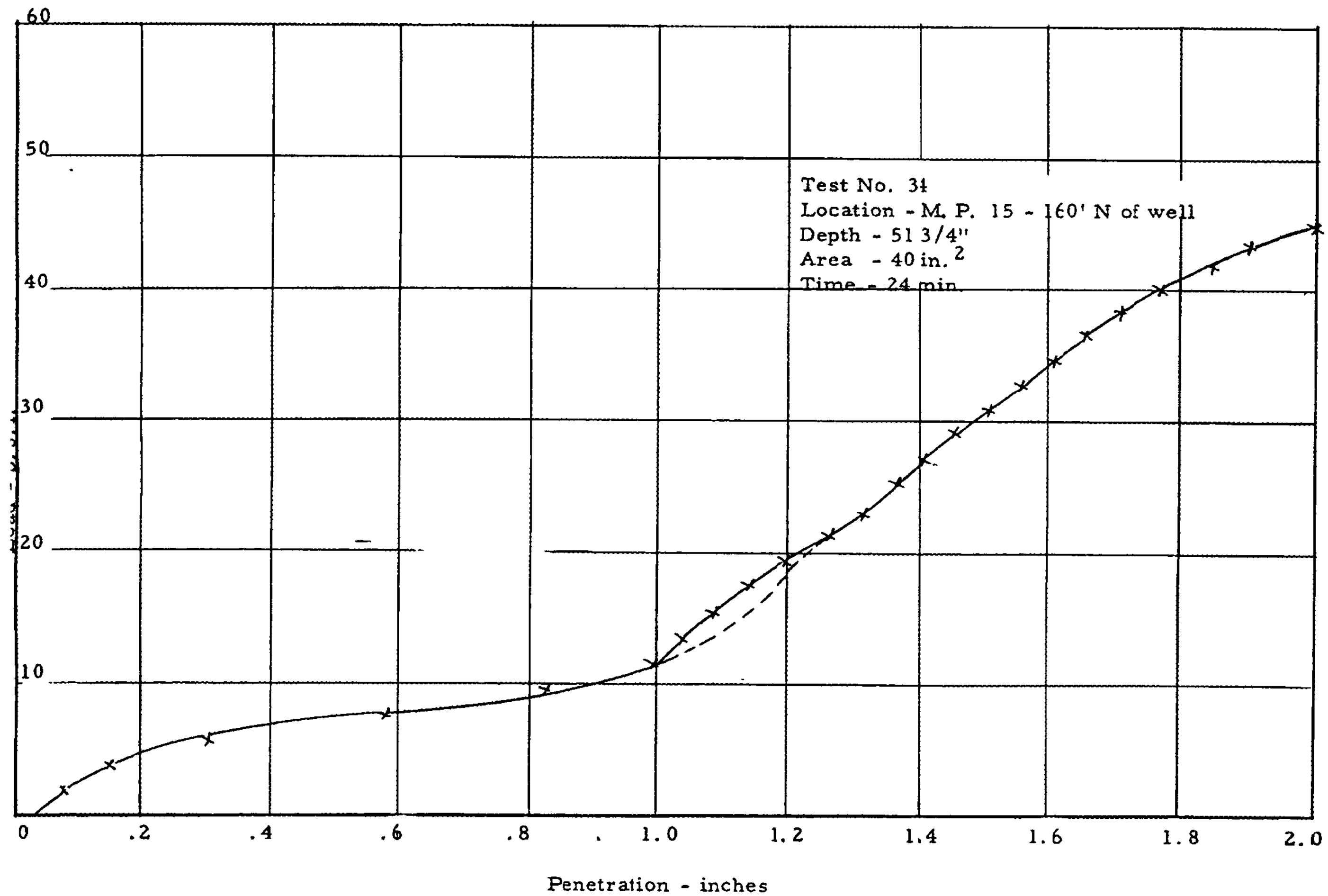


Fig. 9-16. Plate Bearing Tests at Mile Post 15

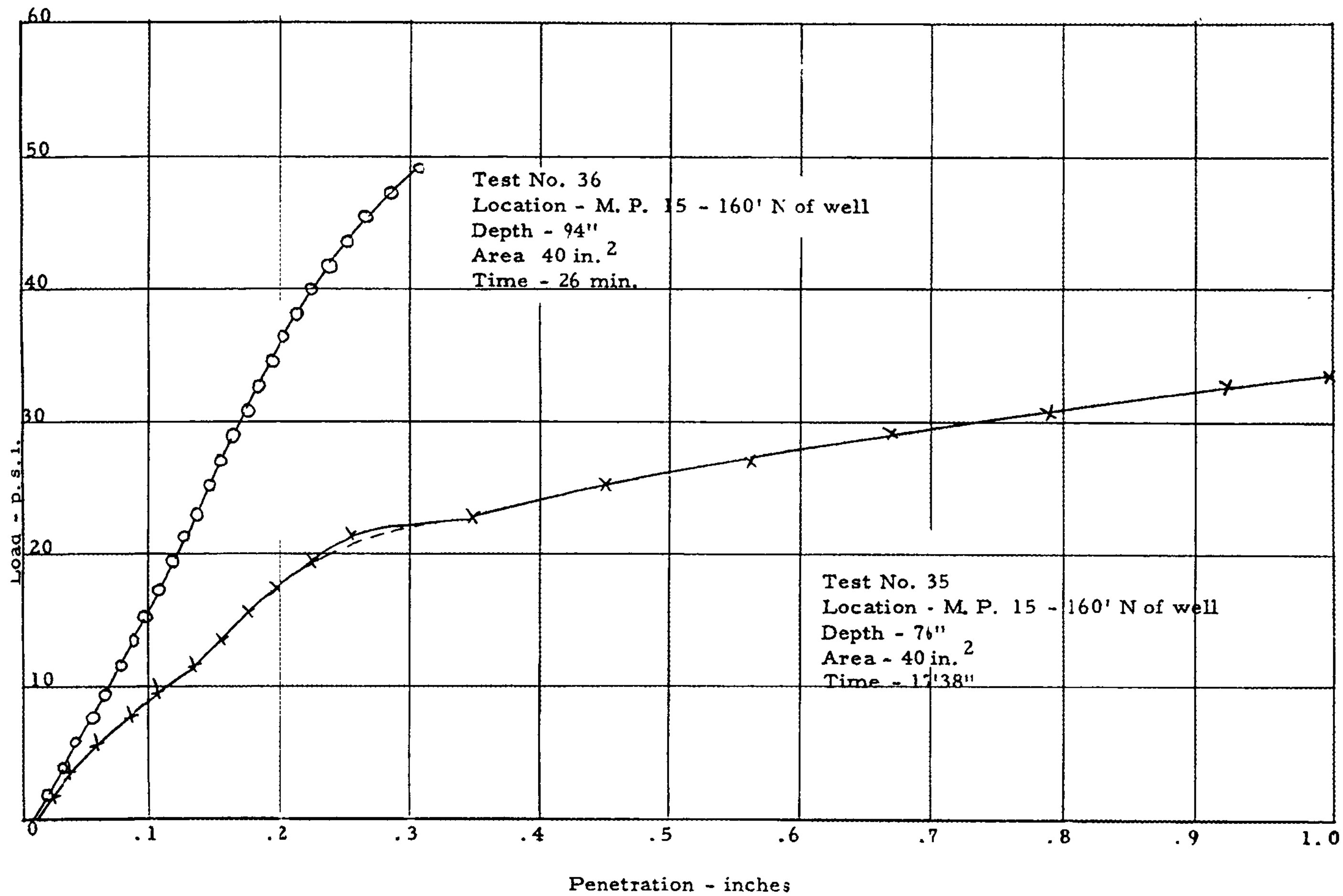


Fig. 9-17. Plate Bearing Tests at Mile Post 15

Table 9-2. Soil data in connection with field plate bearing tests

Test No.	Mile Post mi.	Specific Location	Depth in.	Can No.	Natural Moisture percent	Wet Unit Weight gr/cc	Wet Unit Weight lb/cf	Dry Unit Weight gr/cc	Dry Unit Weight lb/cf
1-2	15	62 ft N of well	0	1	20.05	1.90	118.6	1.58	98.6
3	"	225 ft N of N pole line	0	2	17.62	--	--	--	--
4-11	"	50 ft N of well	0	3	19.93	--	--	--	--
"	"	"	0	4	18.48	--	--	--	--
"	"	"	0	5	19.39	--	--	--	--
"	"	"	0	6	18.91	--	--	--	--
13	3	.5 mi. NW Bonneville Plant	0	7	21.48	1.51	94.2	1.24	77.4
14	3.5	(N of Bonneville Plant	0	8	34.43	1.85	115.4	1.38	86.1
15	"	(midway between pole lines	3.5	9	38.16	1.67	104.2	1.21	75.5
17	11.6	300 ft E of E 25 mi. drain	0	10	39.27	1.79	111.7	1.29	80.5
18	12.65	Midway between pole lines	0	11	27.89	1.96	122.3	1.53	95.5
19	"	"	4.5	12	43.25	--	--	--	--
20	25	100 ft E NE of well	0	13	#	1.63	101.7	--	--
21	"	"	7.5	14	31.24	1.69	105.5	1.27	79.2
23	11.7	(300 ft E of 25 mi. drain,	60	19	38.7	--	--	--	--
24	"	(60 ft S of S pole line	96	20	35.7	--	--	--	--
26	"	50 ft S of test 23 (salt)	0	21	24.9	1.42	88.6	1.13	70.5
27	"	"	10.5	22	38.9				
"	"	"	"	23	37.1	1.89	117.8	1.38	85.9
"	"	"	"	24	36.7				

Apparent error in moisture determination

In what might be described as a normal curve, there is an initial curve, concave upward, then a straight line portion, then a curve concave downward. When the straight line is projected to the axis, there is usually a finite value of the penetration that would correspond to a zero load. This is usually due to the fact that the plate is not perfectly seated and that there is a small initial yield before the plate is fully bearing on the soil. In making the calculations for Table 9-1, corrections were made in the penetration as shown on the curves for this penetration for zero load. For example, in Fig. 9-1 for test 1 the intersection of the straight line portion of the curve and the X-axis is 0.023 inch. The load corresponding to a penetration of 0.2 inch would then be read from the curve for a total penetration of 0.223 inch, and the modulus of deformation as the slope of the curve at this same penetration. To minimize this correction, for most of the tests, it was the practice to apply a small initial load to the plate, then back off the load to a zero load dial reading, then adjust the penetration dial to a zero reading. If this initial load was excessive, the straight line portion of the curve when projected might intersect the X-axis to the left of the origin for a negative penetration reading.

Table 9-1 lists the test number, the specific location of the test and the depth below the surface, the modulus of deformation for three points on the curve, the initial straight line portion, the modulus at a penetration of 0.2 inch and at 0.5 inch, and also the loads corresponding to penetrations of 0.2 and 0.5 inch.'

It will be noticed that some of the curves are not of a typical shape: for example, test 12, Fig. 9-6. Here, after a penetration of about 0.2 inch the curve turns upward instead of continuing downward. This test was made in the 12-foot well used for the transmissibility test.. Before the test was made, the bottom of the hole was flattened with the special bottoming auger constructed for this purpose; but it appears that there was still a layer of soft material in the bottom of the well, underlain with firmer material. The modulus of deformation at a penetration of 1.0 inch is probably more nearly the true modulus for the material below the 12-foot depth than the modulus for the 0.2 and the 0.5-inch penetrations. In summary, a curve that turns upward indicates a firmer material a short distance below the plate, and a curve that breaks sharply to the right as shown for test 23, Fig. 9-11, would indicate a soft material a short distance below the firm material on which the plate was seated. Since the material from the surface to a depth of about 20 feet, as shown by other borings and soil samples, is highly stratified, consisting of layers of soft and firm clays and silts, interspersed with layers of oolitic sand of various thicknesses, and with occasional layers of hardened lime material, or thin salt crusts, it would be surprising if the curves exhibited the properties of a homogeneous material.

At the end of the test, it was the usual practice to discontinue the application of the load allowing the jack handle to remain stationary and to continue the load and penetration readings for several minutes or until the rate of penetration had decreased to about 0.001 inch per minute. The plots of these additional readings are illustrated by the three curves in Fig. 9-7.

Two sets of tests were made to evaluate the effect of size of the plate on the results obtained. Housel* assumes that a load on a foundation, or bearing plate, is supported partly by pressure on the plate area, and partly by perimeter shear. The relationship has been expressed by the equation

$$L = n A + m P \qquad (1)$$

where: L = total land in pounds
A = area of the plate
P = perimeter of the plate
n and m = coefficients that can be evaluated by testing plates of two sizes and solving the simultaneous equations.

For tests 8, 9, and 10, Fig. 9-5, plate areas of 20, 40, and 80 square inches were used. These tests were made on the surface near each other. For a penetration of 0.5 inch, the three equations can be written:

* W. S. Housel, A Practical Method for the Selection of Foundations Based on Fundamental Research in Soil Mechanics. University of Michigan Eng. Res. Bul. 13, Oct. 1929.

$$484 = 20\ n + 15.87\ m \quad (2)$$

$$880 = 40\ n + 22.44\ m \quad (3)$$

$$1384 = 80\ n + 31.74\ m \quad (4)$$

Solving equations (2) and (3) simultaneously, one obtains:

$n = 16.7$ and $m = 9.5$

For equations (2) and (4), the values are:

$n = 10.4$ and $m = 17.4$

For equations (3) and (4), the values are:

$n = 6.2$ and $m = 28.1$

The agreement is not good. Using the mean value of $n = 11.1$ and $m = 18.3$, the calculated and actual loads on the three plates for a penetration of 0.5 inch would be:

Plate Area Square inches	Calculated Load pounds	Actual Load pounds
20	512	484
40	855	880
80	1469	1384

Similar comparisons can, of course, be made for other penetration values.

The second set of tests on the different size plates at the same location were tests 29, 30, 31, and 32 shown in Fig. 9-14. Here, six sets of simultaneous equations can be set up and solved. For a penetration of 0.5 inch, the solution of these equations results in the following values of n and m.

Plate Area square inches	Value of n psi	Value of m psi
10 and 20	23.59	3.04
10 and 40	14.01	11.58

(Continued)

Plate Area square inches	Value of n psi	Value of m psi
10 and 80	11.54	13.78
20 and 40	7.23	23.65
20 and 80	8.00	22.68
40 and 80	8.55	21.30
Mean	12.15	16.00

Plate Area square inches	Calculated Load pounds	Actual Load pounds
10	301	270
20	497	520
40	845	820
80	1480	1300

Tests 33, 34, 35, and 36 and Figs. 9-15, 9-16, and 9-17 were made for the purpose of evaluating the bearing capacity of the subsoil, especially the blocky clay in which the subsoil flow takes place. These tests were made at depths of 40.5, 51.75, 76, and 94 inches below the surface. Tests 33, at a depth of 40.5 inches, was in a soft clay below a sand layer. The initial modulus of deformation was fairly high, 203.3 psi per inch, but at a load of 10 psi, the soil began to yield, and the modulus dropped to a low value. At a depth of 51.75 inches, test 34, the initial modulus was very low and the load was only about 5 psi for a penetration of 0.2 inch. At a penetration of 0.1 inch, the load was only 12 psi. The penetration dial was reset and the test continued. The time delay with the continued penetration of the plate while the dial was being reset caused a sharp break in the curve. The probable curve, had there been no interruption, is shown by a dotted line. It appears from this curve that the plate was on a very soft clay with considerably firmer material a short distance below. For test 35, at a depth of 76 inches, the plate was bearing on a layer of oolitic sand of several inches in depth. This material had a higher bearing capacity but began to yield at a load of about 20 psi. At the final depth of 94 inches, the plate was on a firm clay below the sand. Here the modulus of deformation and load bearing capacity was much greater. These tests illustrate the highly stratified and variable nature of the subsoil materials and the difficulty that might be experienced in trying to predict accurately the settlement of the fill that might be expected. The soft material found at the 51.75-inch depth might yield and consolidate considerably under the load induced by the fill and its placement. Just what the stabilizing effect of the firmer materials overlying this soft material would be cannot be predicted accurately. The existing load of the overlying materials at a depth of 52 inches would be about 3 psi. When a fill load of about 5 psi is added it would be well past the yield point for this soil.

Conclusions

1. A technique has been worked out for making a field plate bearing tests that yield useful data on the bearing capacity of the soil.

2. A complete test can be made in a period of about one hour, permitting several tests to be made in a day.

3. The tests can be made on the undisturbed surface, or at any desired depth below the surface. Tests were successfully made at a depth of 12 feet, but the same technique could be used for depths up to 20 feet or more. The principal difficulty at such a depth would be that of constructing the hole with a hand auger.

4. The tests performed on the surface indicate that the bearing capacity as indicated by the load at a given penetration of the plate, or by the modulus of deformation at any given penetration, depends upon the plate area, and that some of the load is carried by perimeter shear.

5. Tests performed at different depths below the surface show the highly stratified and variable nature of the soil with depth.

6. It is believed from a study of the test data obtained to date that the soils in the areas tested has an adequate bearing capacity for low fills of the order of about 5 feet, but that considerable consolidation and settlement might take place.

7. It would appear doubtful that actual settlement could be predicted from such tests without first correlating the results of rather detailed field tests with subsequent settlement data. For the benefit of future investigations, it might be very desirable to conduct such a detailed study in a given area and follow up with careful determinations of the settlement of the fill over a period of time.

8. Long-time plate bearing tests, in which the plate of somewhat larger area would be loaded and left in place until no further settlement occurred, might yield very useful data, especially if such tests were conducted in connection with short-time tests of the type described in this report. Equipment for such long time-tests has not yet been developed.

Part X

TRANSMISSIBILITY TESTS -- 1961

by

J. E. Christiansen

Bal B. Patil

Introduction

On July 5 and 6, and 10-12, 1961, additional transmissibility tests were attempted at seven locations beginning at mile post 15 and extending to mile post 40 at Knolls. With the exception of the test at mile post 17 all locations were at five mile intervals.

Wells were bored with an eight inch auger to a depth of about nine feet at all locations. Soil samples were taken at all sites for laboratory tests.

The first pumping test was made at mile post 15. The results were quite similar to those obtained at Stations 392+91 and 444+70 (approximately mile posts 7.4 and 8.4) as reported in Part II. The transmissibility values at these two stations were the highest found along the route between mile posts 7.4 and 12.2 The second pumping test was attempted at mile post 20. The transmissibility at this location proved to be so low that the pumping method could not be used. A pumping test was then attempted at mile post 17, but again the transmissibility was too low to obtain satisfactory results.

At no place east of mile post 15 was the transmissibility sufficient to obtain satisfactory results with the pumped-well method. Tests were made at mile posts 17 and 20 by the post-hole method for determining permeability. At mile posts 25, 30, 35, and 40, the water entered the hole so slowly that tests by the post-hole method were not feasible, and no actual tests were attempted.

Test at Mile Post 15

The tests at mile post 15 were made on July 5, the well having been bored to about 8.7 feet the previous day. Piezometers were installed at distances of 10, 20, 50, 100, and 200 feet from the well in all four cardinal directions. All piezometers were 10.5 feet in length. They were driven to an average depth of 8.7 feet. In driving the piezometers it was noted that hard layers of material were encountered at depths of 2.1, 5.7, and 8.6 feet below the surface. Above the first hard layer and between the first and second hard layers, the piezometers penetrated very easily and could be pushed by hand. Heavy blows with the piezometer hammer were required to penetrate the hard layers.

Three pumping tests were made at mile post 15 on July 5. The results of the tests are summarized in Tables 10-1 to 10-3. Tests 1 and 2 were made with the well depth of 8.7 feet; test 3 was made after deepening the well to 12.2 feet.

All values of the transmissibility, T, were computed from the equi-

Table 10-1. Transmissibility at Mile Post 15, Test 1

Radius, r	Elevation of Water in Piezometers, h South	North	West	East	Average	r_2/r_1	h_2-h_1	Transmissibility
feet	feet	feet	feet	feet	feet		feet	cfs/ft
10	7.43	7.30	7.37	7.46				
	7.35	7.25	7.31	7.44	7.36			
						100/10	0.51	0.122
20	7.62	7.52	7.47	7.50				
	7.53	7.46	7.43	7.49	7.50			
						100/20	0.37	0.118
50	7.76	7.82	7.75	7.76	7.73			
						200/20	0.46	0.135
100	7.87	7.94	7.84	7.85	7.87			
						200/10	0.60	0.135
200	7.94	8.01	7.96	7.94	7.96			
						Average		0.128

Average time after starting pump, 20 minutes

Average discharge of pump, 0.170 cubic feet per second

Depth of well, 8.7 feet

Table 10-2. Transmissibility at Mile Post 15, Test 2

Radius r	Elevation of Water in Piezometers, h South	North	West	East	Average	r_2/r_1	$h_2 - h_1$	Transmissibility
feet	feet	feet	feet	feet	feet		feet	cfs/ft
10	7.19	7.09	7.15	7.30				
	7.20	7.09	7.14	7.28	7.18			
						100/10	0.48	0.114
20	7.34	7.30	7.26	7.33				
	7.35	7.29	7.25	7.33	7.31			
						100/20	0.35	0.110
50	7.50	7.58	7.54	7.48	7.53			
						200/20	0.49	0.112
100	7.60	7.71	7.66	7.65	7.66			
						200/10	0.62	0.115
200	7.79	7.85	7.81	7.74	7.80			
						Average		0.113

Average time after starting pump second time, 115 minutes

Average discharge of pump, 0.150 cubic feet per second

Depth of well, 8.7 feet

Table 10-3. Transmissibility at Mile Post 15, Test 3

Radius r	Elevation of Water in Piezometers, h South	North	West	East	Average	r_2/r_1	h_2-h_1	Transmissibility
feet	feet	feet	feet	feet	feet		feet	cfs/ft
10	6.92	6.76	6.62	7.11				
	6.88	6.76	6.62	7.12	6.85			
						100/10	0.78	0.071
20	7.18	7.04	7.04	7.14				
	7.17	7.03	7.03	7.14	7.10			
						100/20	0.53	0.073
50	7.43	7.52	7.45	7.40	7.45			
						200/20	0.67	0.083
100	7.58	7.69	7.65	7.61	7.63			
						200/10	0.92	0.079
200	7.76	7.82	7.81	7.70	7.77			
						Average		0.076

Average time after starting pump third time, 129 minutes

Average discharge of pump, 0.152 cubic feet per second

Depth of well, 12.2 feet

librium equation for artesian wells:

$$T = \frac{0.366\, Q \log r_2/r_1}{(h_2 - h_1)}$$

There seems to be no logical explanation for the lower transmissibility values for the third test. One possible explanation might be that there are two or more strata through which the water moves readily and that the piezometers are terminated in only one of these, or possibly in none of them, and do not, therefore, indicate the true drawdown water surface for the well. While deepening the well, one or more of the strata could have been "muddied" up, impeding the flow into the well, but the piezometers might not have reflected the drawdown for this stratum or these strata. A change in the flow into the well would not then be reflected by a corresponding change in the drawdown as indicated by the piezometers.

The difference in the transmissibility values for the three tests is of no practical importance. All three tests show that the blocky clay structure through which the flow occurs is present at this location, and that the transmissibility is as high here as at any other location for which pumping tests were made. Visual evidence, however, indicates that some of the water was entering the well at a higher level than was generally the case at the other locations.

The initial slope of the water table (prior to the tests), as computed from the readings on the 200-foot radius piezometers, is given in Table 10-4. It would appear from this table that there may have been small errors in the static readings on the 50- and 100-foot radius piezometers to the west. Neither of these readings enter into the calculations of the initial slope.

Tests at Mile Post 17 and 20

Because of the very low transmissibility at mile posts 17 and 20, pumping tests were not feasible. Permeability tests were made, however, by the auger-hole method as described by Winger.*

The average value of the permeability, K, expressed in inches per hour, is calculated from the test data by the method outlined. These values have also been converted to transmissibility values by multiplying by the static depth of water in the hole prior to the test. The results of the tests are summarized briefly in Table 10-5.

* R. J. Winger, Jr. "In-Place Permeability Tests and Their Use in Subsurface Drainage." Prepared for the International Commission on Irrigation and Drainage, Fourth Congress, Madrid, Spain. June 1960.

Table 10-4. Initial Slope of Water Table at Mile Post 15

Radius r	Elevation of Static Water Level in Piezometers, h South	North	West	East	Average	$(h_N - h_S)$	$(h_E - h_W)$
feet	feet	feet	feet	feet	feet	feet	feet
10	7.96	8.02	8.01	8.02	8.00	0.06	0.01
20	8.01	8.02	8.01	8.02	8.01	0.01	0.01
50	8.01	8.02	8.06	8.03	8.03	0.01	-0.03
100	8.00	8.03	7.99	8.02	8.01	0.03	0.01
200	7.79	8.06	8.01	8.03	8.02	0.09	0.02

Considering only the 200-foot radius piezometers:

$$\text{Slope} = \frac{\sqrt{(h_N - h_S)^2 + (h_E - h_W)^2}}{400}$$

$$= \frac{\sqrt{(0.09)^2 + (0.02)^2}}{400}$$

$$= \frac{\sqrt{0.0081 + 0.0004}}{400}$$

$$= \frac{\sqrt{0.0085}}{400} = \frac{0.092}{400} = 0.00023 \text{ or } 0.23 \text{ feet per 1000 feet}$$

Direction of greatest slope = South 16° West

Table 10-5. Permeability Tests by the Auger-Hole Method

Mile Post	Test No.	Depth of Well	Diameter of Well	Permeability K	Transmissibility T
		feet	inches	in/hr	cfs/ft
17	1	6.5	4	1.45	0.00018
	2	6.5	4	1.14	0.00015
	3	9.4	4	1.70	0.00033
	4	9.4	4	0.64	0.00015
20	1	12.0	8	0.70	0.00021
	2	12.0	8	0.51	0.00014
	3	12.0	8	0.58	0.00016

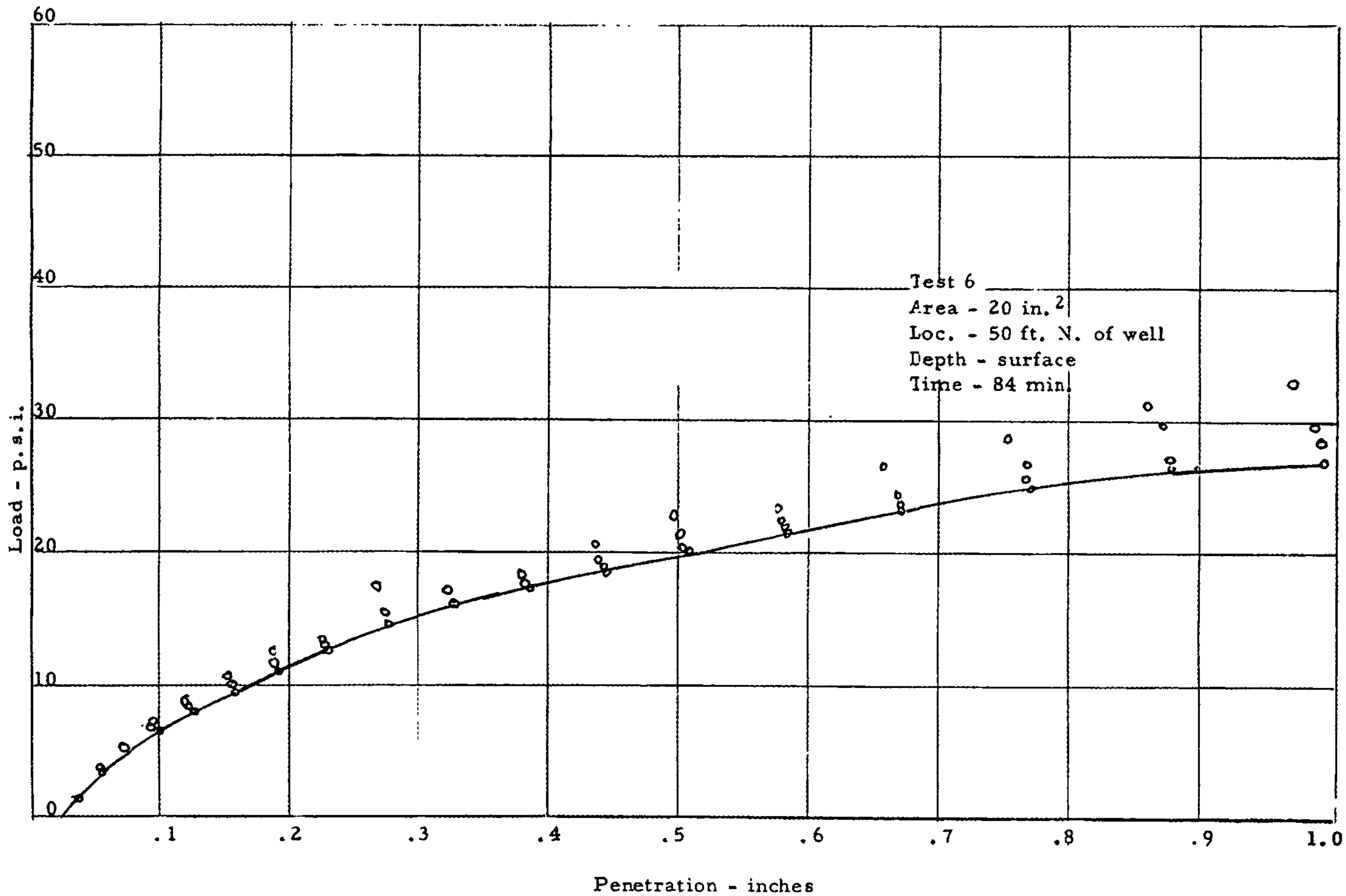


Fig. 9-3. Plate Bearing Test at Mile Post 15

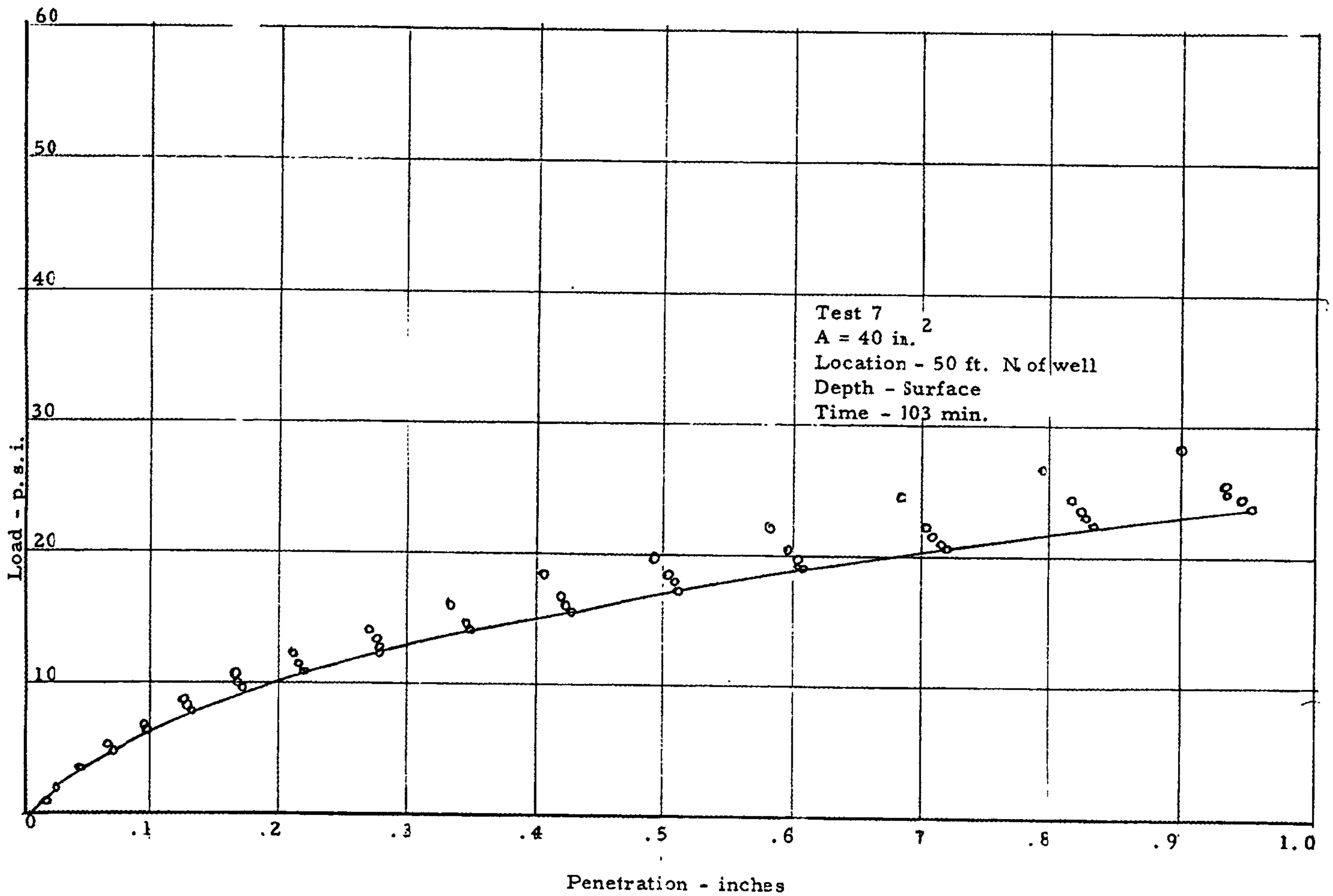


Fig. 9-4. Plate Bearing Test at Mile Post 15

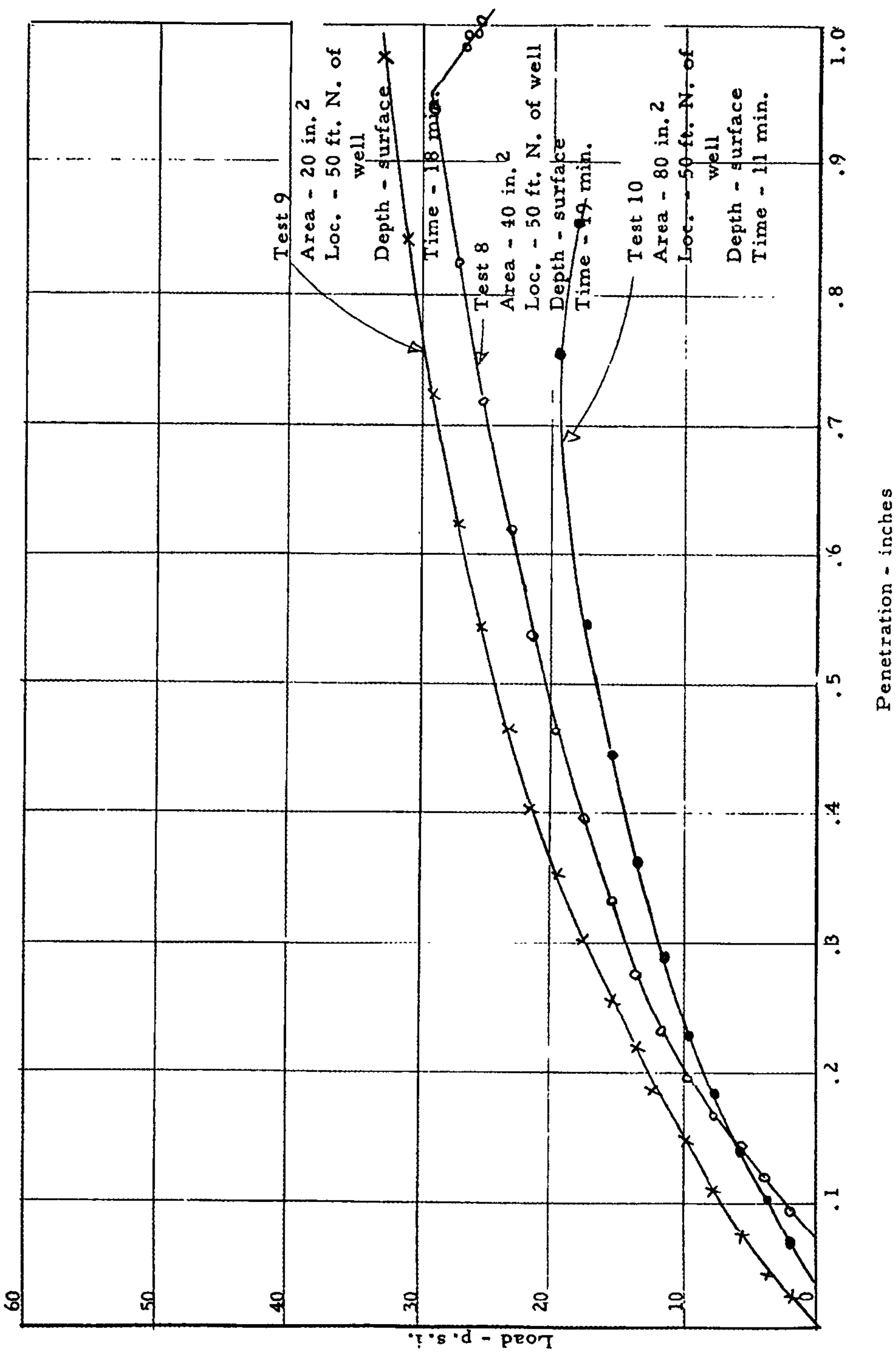


Fig. 9-5. Plate Bearing Test at Mile Post 15

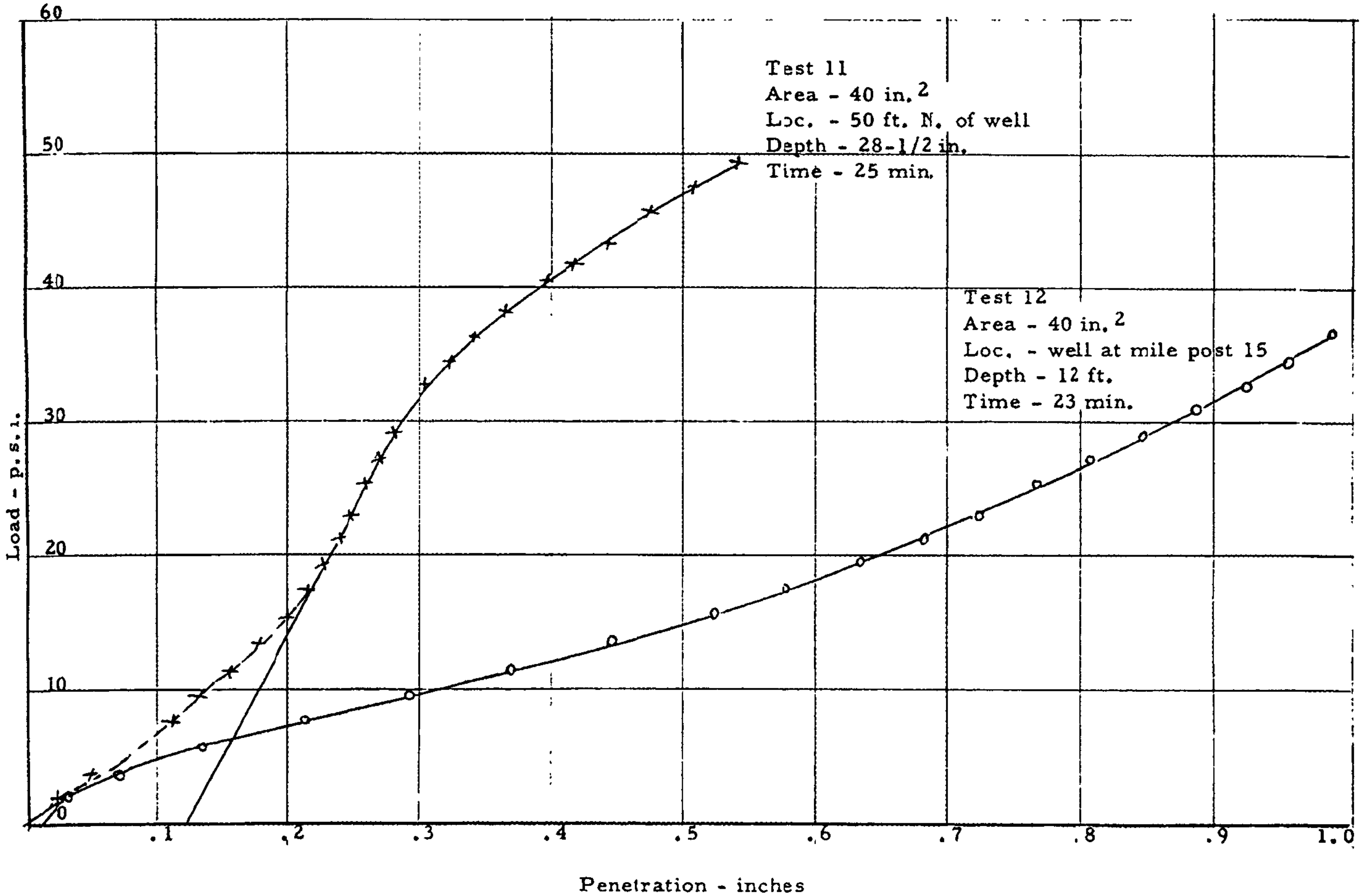


Fig. 9-6. Plate Bearing Test at Mile Post 15

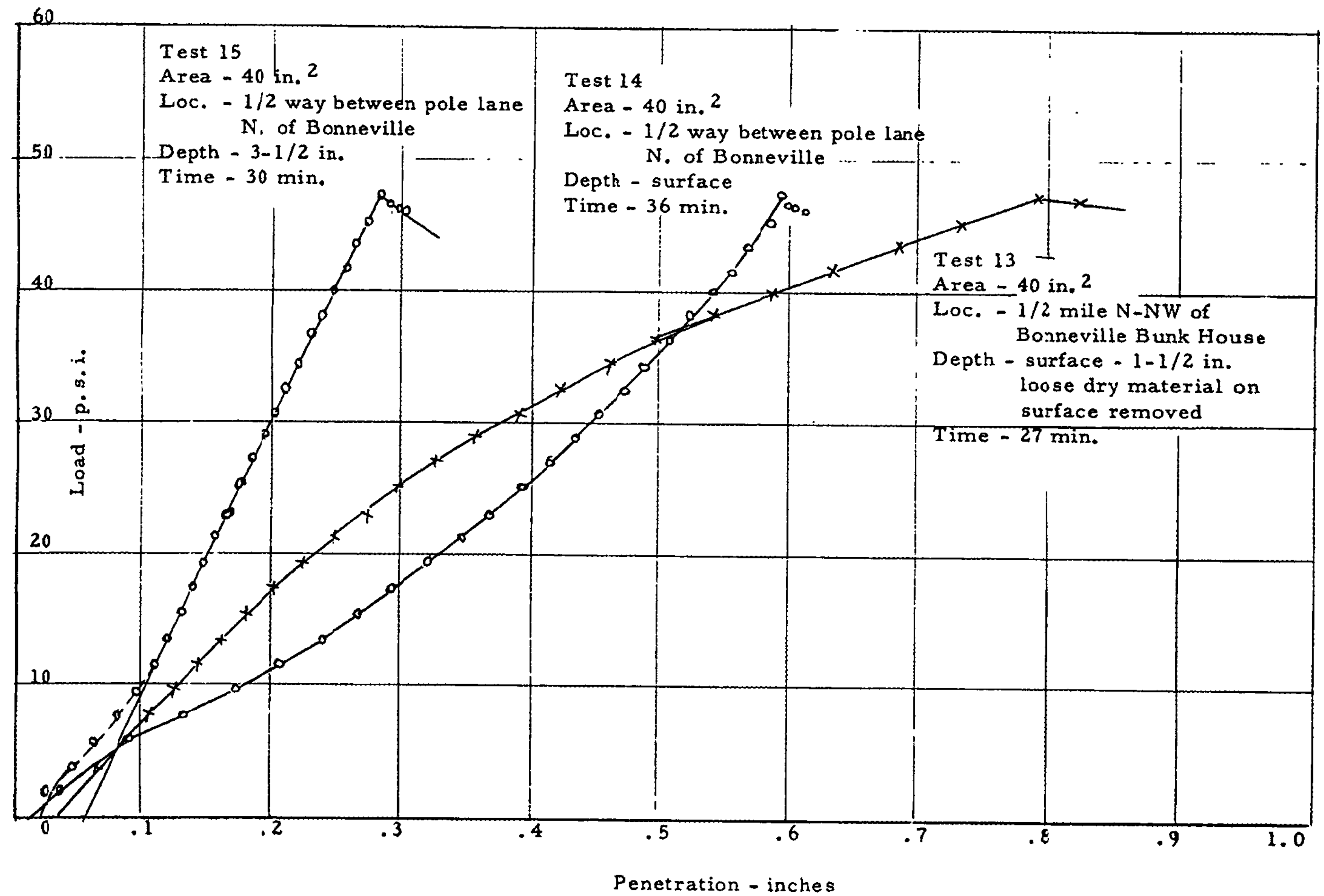


Fig. 9-7. Plate Bearing Tests Near Bonneville Bunk House

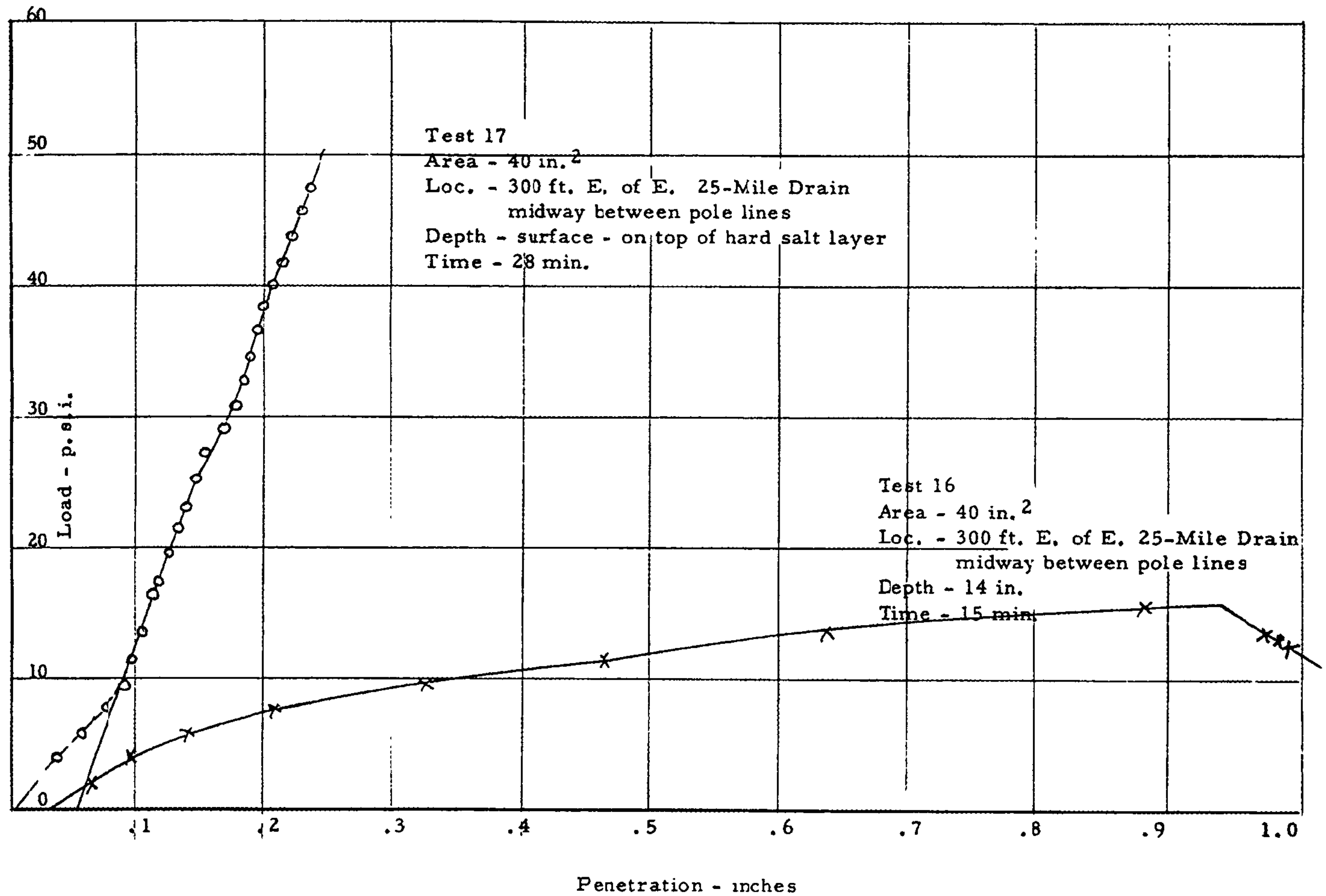


Fig. 9-8. Plate Bearing Tests at 25-Mile Drain

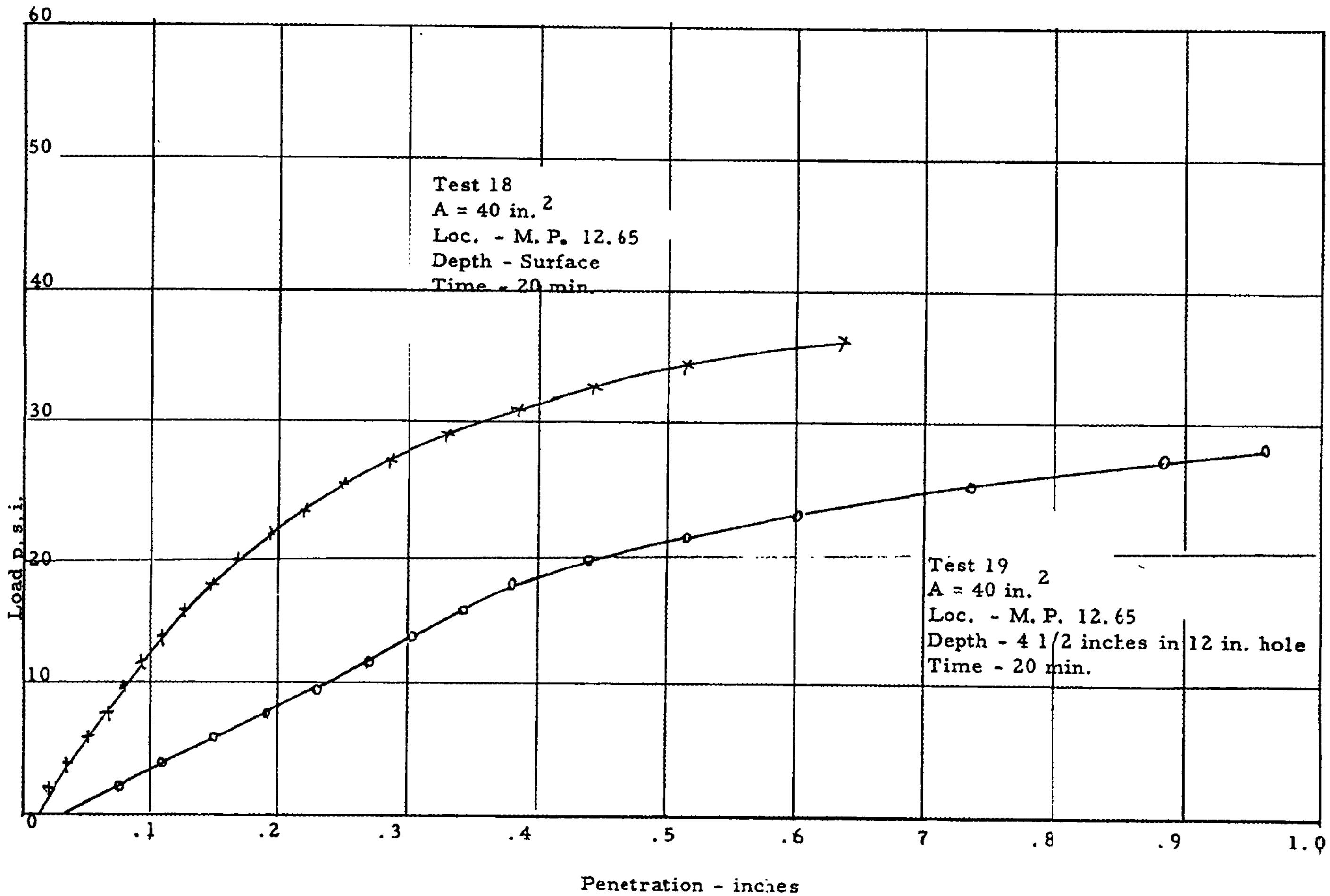


Fig. 9-9. Plate Bearing Tests at Mile Post 12.65

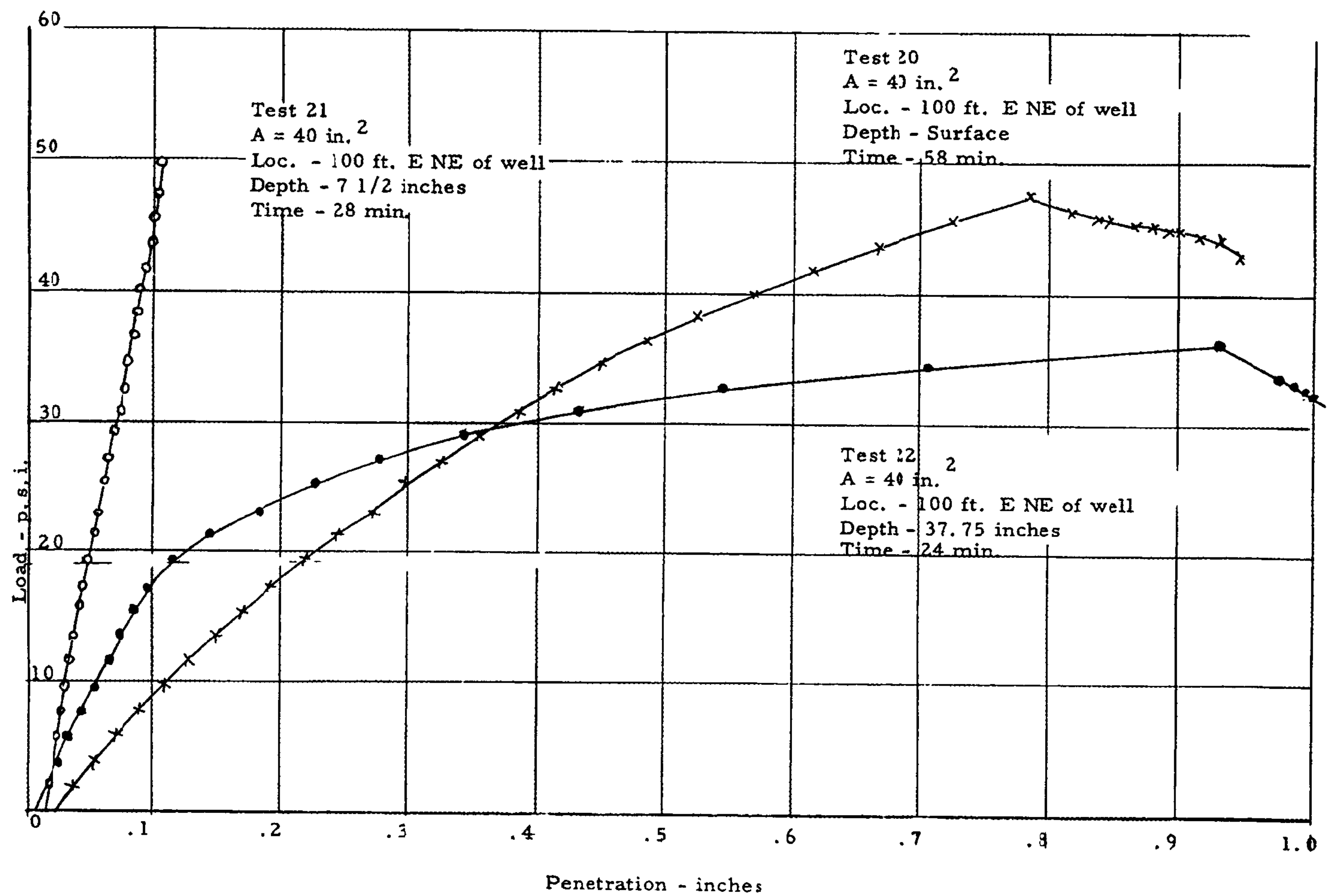


Fig. 9-10. Plate Bearing Tests at Mile Post 25

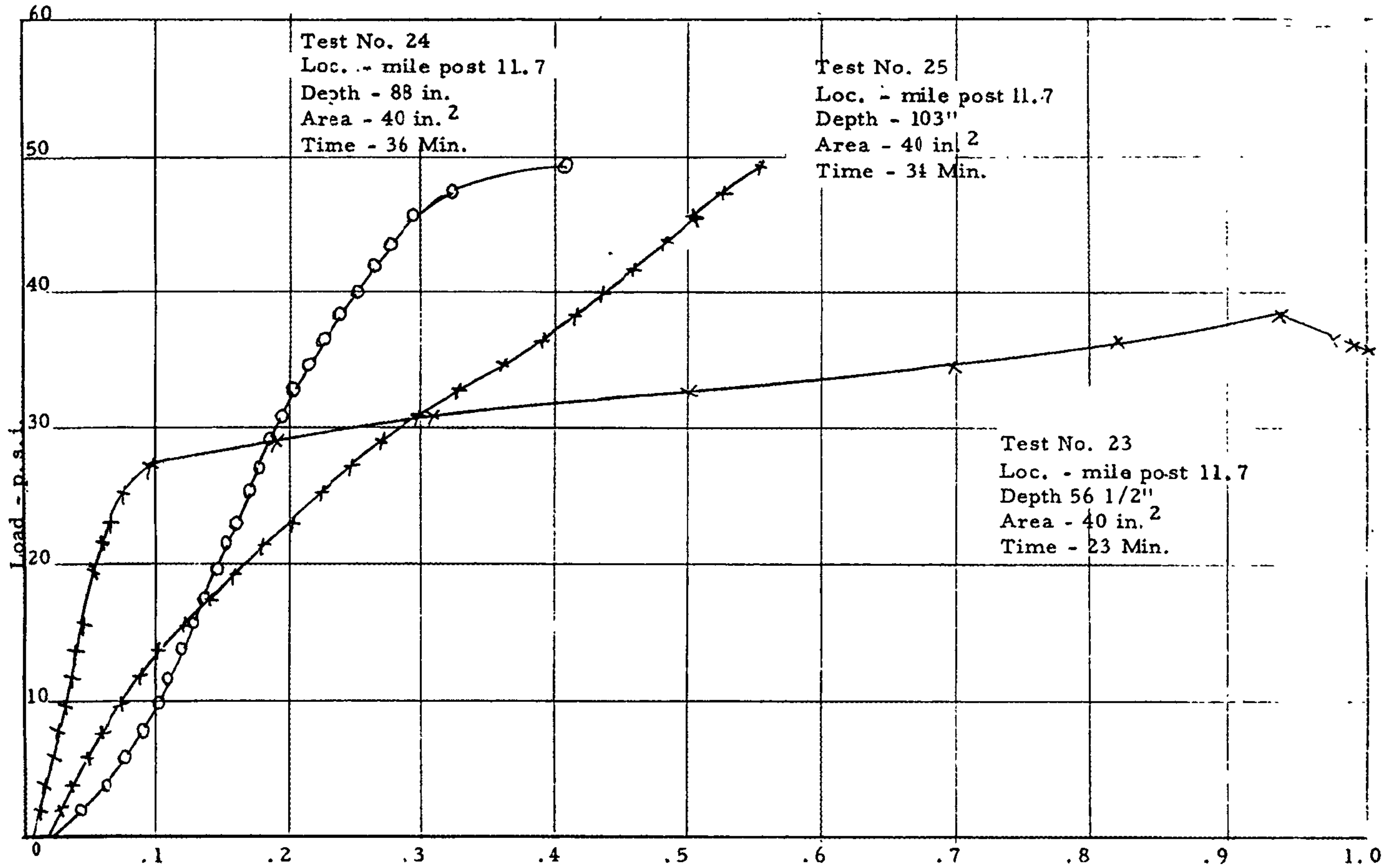
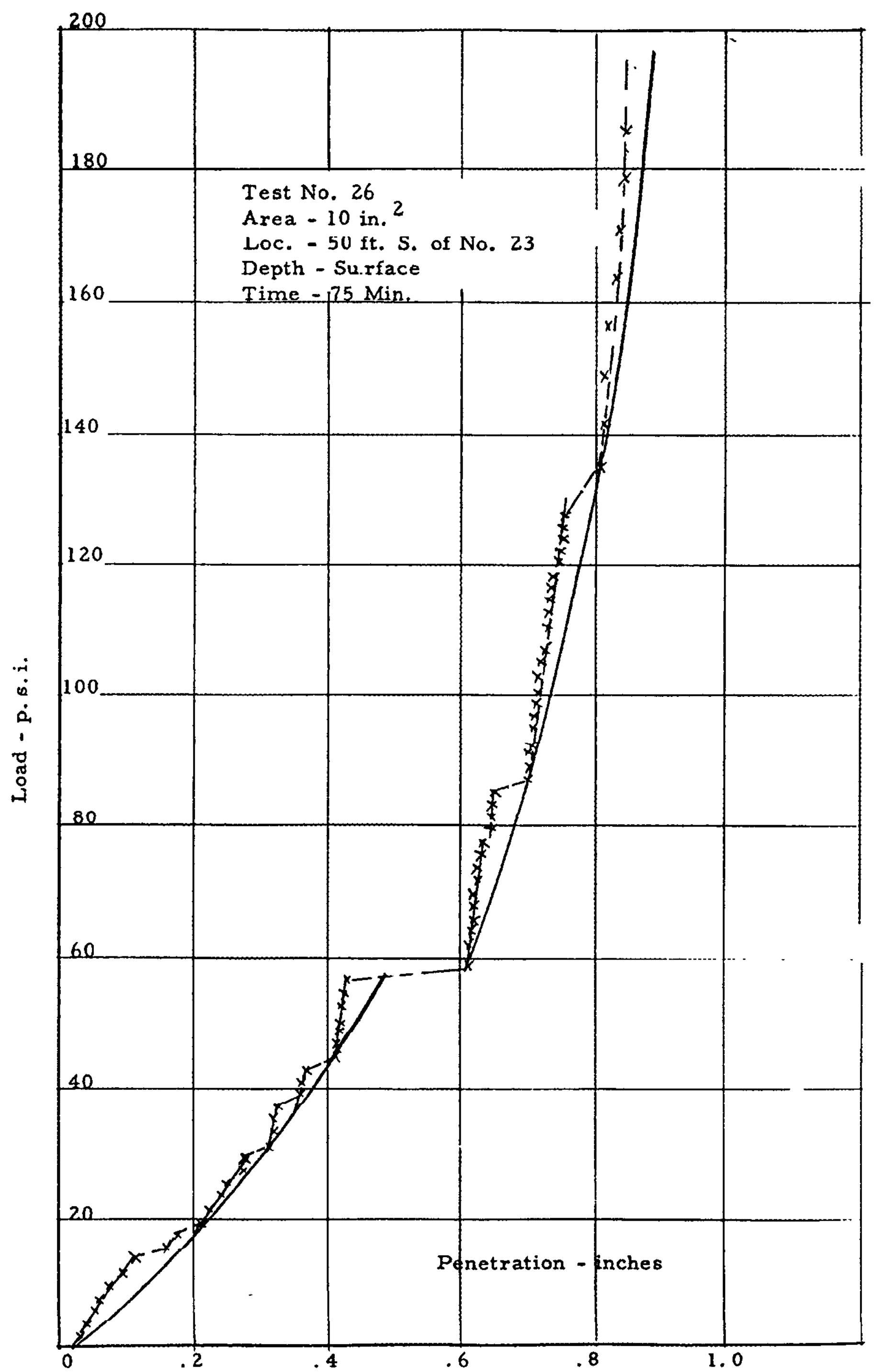


Fig. 9-11, Plate Bearing Tests at Mile Post 11.7 near 25-Mile Drain


Test No. 26
Area - 10 in.^{2}
Loc. - 50 ft. S. of No. 23
Depth - Surface
Time - 75 Min.
Load - p.s.i.
Penetration - inches
200
180
160
140
120
100
80
60
40
20
0
.2
.4
.6
.8
1.0

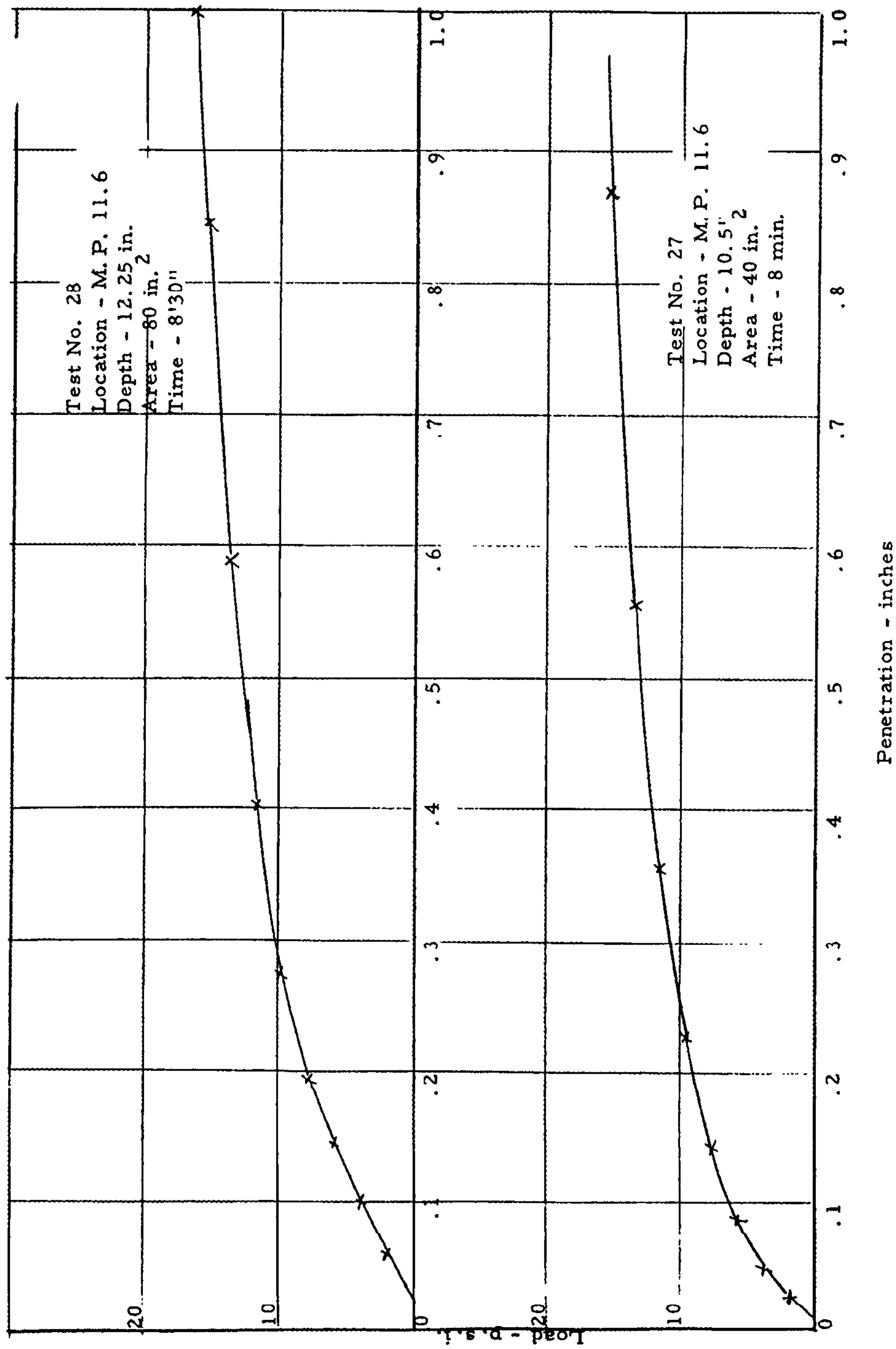


Fig. 9-13. Plate Bearing Tests at Mile Post 11.6

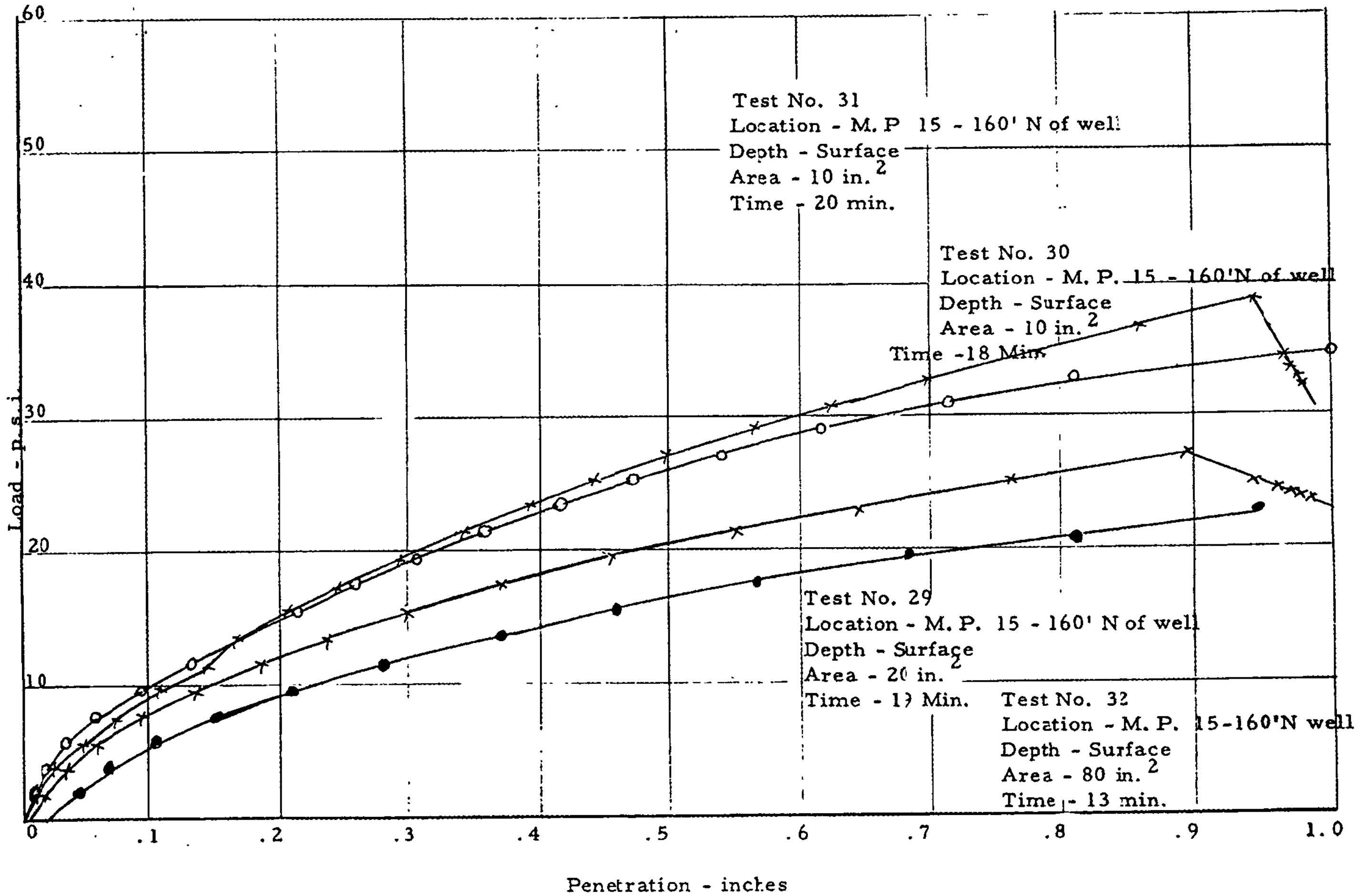


Fig. 9-14. Plate Bearing Tests at Mile Post 15

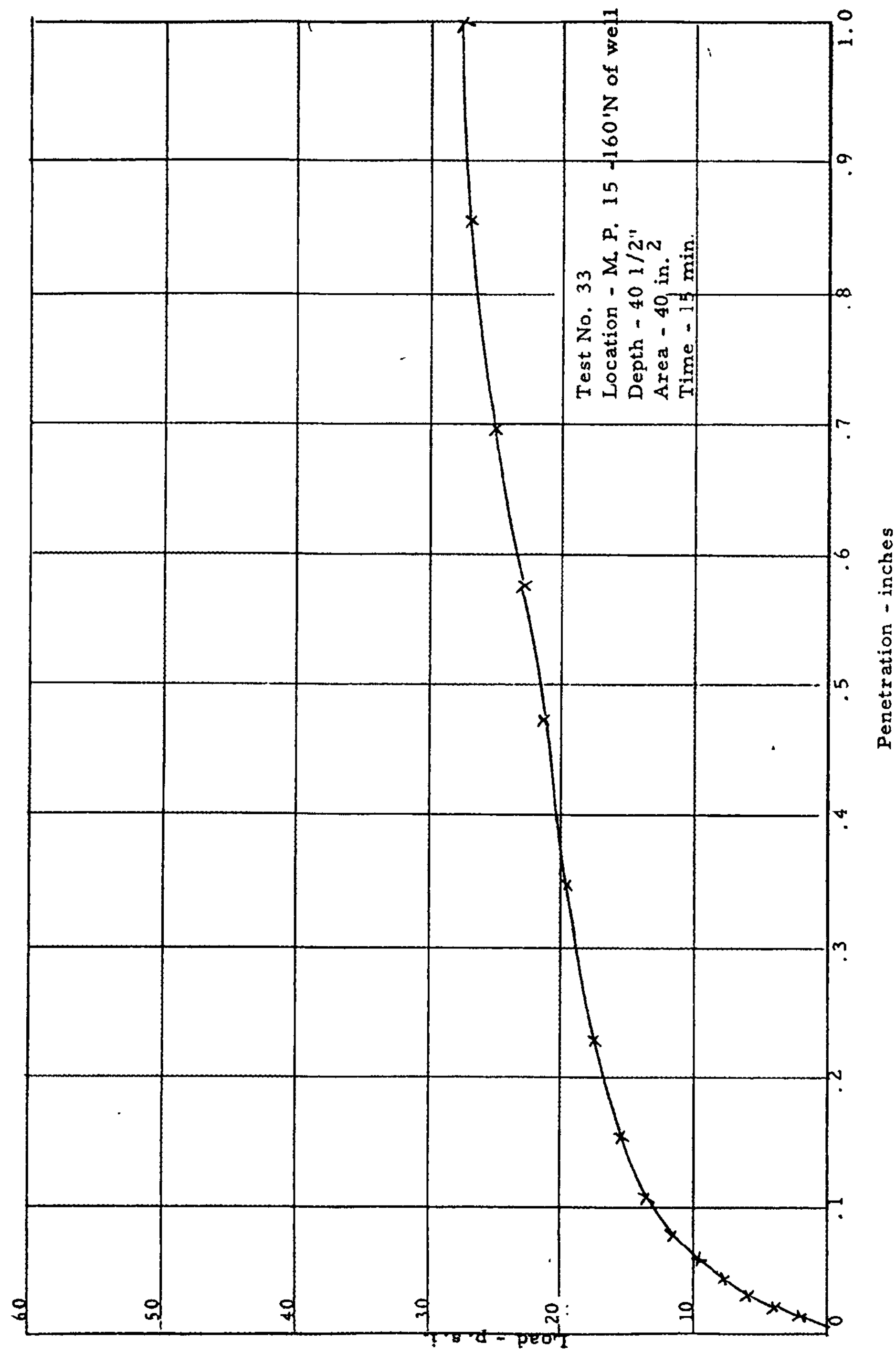


Fig. 9-15. Plate Bearing Tests at Mile Post 15

The lower values for tests 2 and 4 as compared with tests 1 and 3 at mile post 17, and for tests 2 and 3 as compared with test 1 at mile post 20, are believed to be caused by insufficient time for the water level to reach a true equilibrium between the tests. The static water level in the wells at both locations prior to the first test was used as the static water level in the computations for all tests even though sufficient time did not elapse between tests for the water level to again reach this initial level.

It had been anticipated that both methods would be compared by making both types of tests on the same well. Unfortunately, the difference in permeability and transmissibility at mile post 15 as compared with mile posts 17 and 20 made this impractical. The transmissibility at mile post 15 was so high that tests by the auger hole method were impossible. Likewise, the permeability at mile posts 17 and 20 was so low that pumping tests were impractical.

Summary and Conclusions

1. The transmissibility values at mile post 15 were very high, approximately the same as at mile post 7.4 and higher than at any of the other eight intermediate locations tested.

2. No increase in transmissibility was achieved by deepening the well from 8.7 to 12.2 feet. Actually, an unaccountable decrease in the transmissibility resulted from this deepening.

3. The transmissibility as computed from the permeability tests by the auger-hole method at mile posts 17 and 20 was the order of magnitude of 1/500 of the value for mile post 15. This indicates that the highly permeable blocky clay with vertical dessication cracks does not extend as far east as mile post 17.

4. Extremely low transmissibility values were apparent from the rate at which the auger holes filled at mile posts 25, 30, 35, and 40. These holes filled so slowly that even the auger-hole method was not practical for determining the transmissibility.

Part XI

ANALYSIS OF STRUCTURAL BEHAVIOR OF THE SALT LAYER AT BONNEVILLE SALT FLATS

by

Reynold K. Watkins

Dwayne Nielson

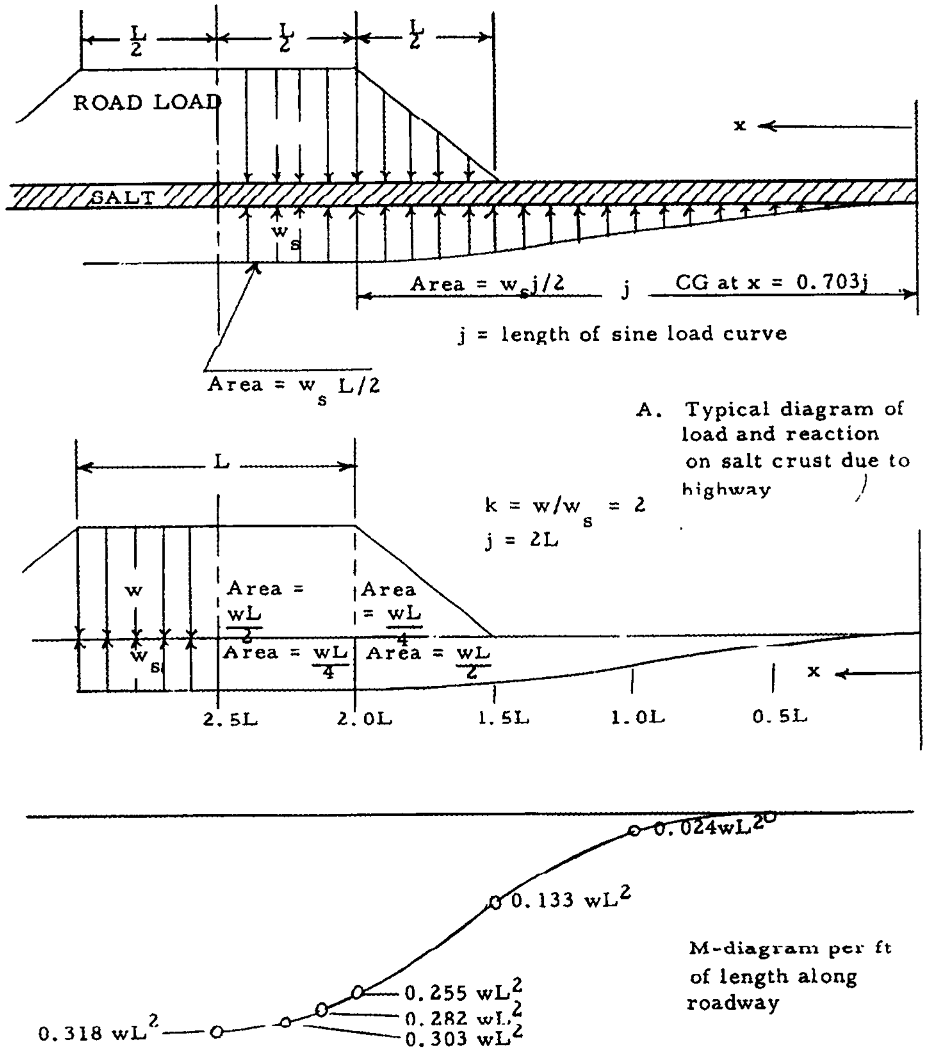
Introduction

The object of the analysis is to determine if the Bonneville salt crust can be depended upon as a structural sub-base slab to support the inter-state highway.

It is reported that the salt crust can support remarkably high loads. For example, an automobile may be driven on a salt crust no more than one inch thick, even though the soil immediately beneath it may not appear to have sufficient bearing strength to support the tire loads. It might seem reasonable, then, that a salt crust many inches thick (up to 40) could surely support a highway. Such is not the case.

Analysis

First consider the flexural strength of the salt crust, assuming it to be a one-way slab beneath the highway. It will be assumed initially that the salt crust is perfectly elastic and that it will retain constant structural properties with time. Since the salt is "floating" on a lake of mud, it can only develop and support a load if it deforms as a loaded beam. The deflection of the salt is dependent not only upon the stiffness of the salt crust (modulus of elasticity and thickness of the salt) but also upon the behavior of the underlying soil. According to unconfined compression tests on the underlying clay the load bearing capacity is not less than about 10 psi. The underlying silt and sand is of no concern as it will be confined and can carry much more than 10 psi. Assuming a typical road cross section, the pressure distribution on the top and bottom of the salt will appear somewhat as shown in Fig. 11-1. The shapes of these load diagrams are assumed; nevertheless, they are entirely reasonable. Note that the sub-base reaction diagram feathers off to zero according to a sine curve. The area under the sub-base reaction diagram must be equal to the area under the road load pressure diagram in order for equilibrium to exist. Under this loading condition, the maximum moment in the salt occurs at the center line of the road. If the road load, w, is assumed to be twice the sub-base reaction, ws, the maximum moment in the salt crust is $M = 0.318\ wL^2$ (see Fig. 11-2). For w = 700 lb per sq ft and L = 36 ft, M = 3,460,000 lb in. For a salt crust 20 inches thick, the maximum stress would be 4330 psi. If the road load, w, is assumed to be 4/3 times the sub-base reaction, w_s, the maximum moment in the salt crust is M = 1,100,000 lb in, and the maximum stress in a 20-inch thick salt layer would be 1375 psi. The use of 20 inches for salt crust thickness in the above analysis is reasonable even though the maximum salt thickness may be as



A. Typical diagram of load and reaction on salt crust due to highway

B. The moment diagram for the idealized load of Figure A.

Figure 11-1. Load and moment diagrams for salt crust assuming the vertical road load is twice as great as the bearing capacity of the underlyi soil

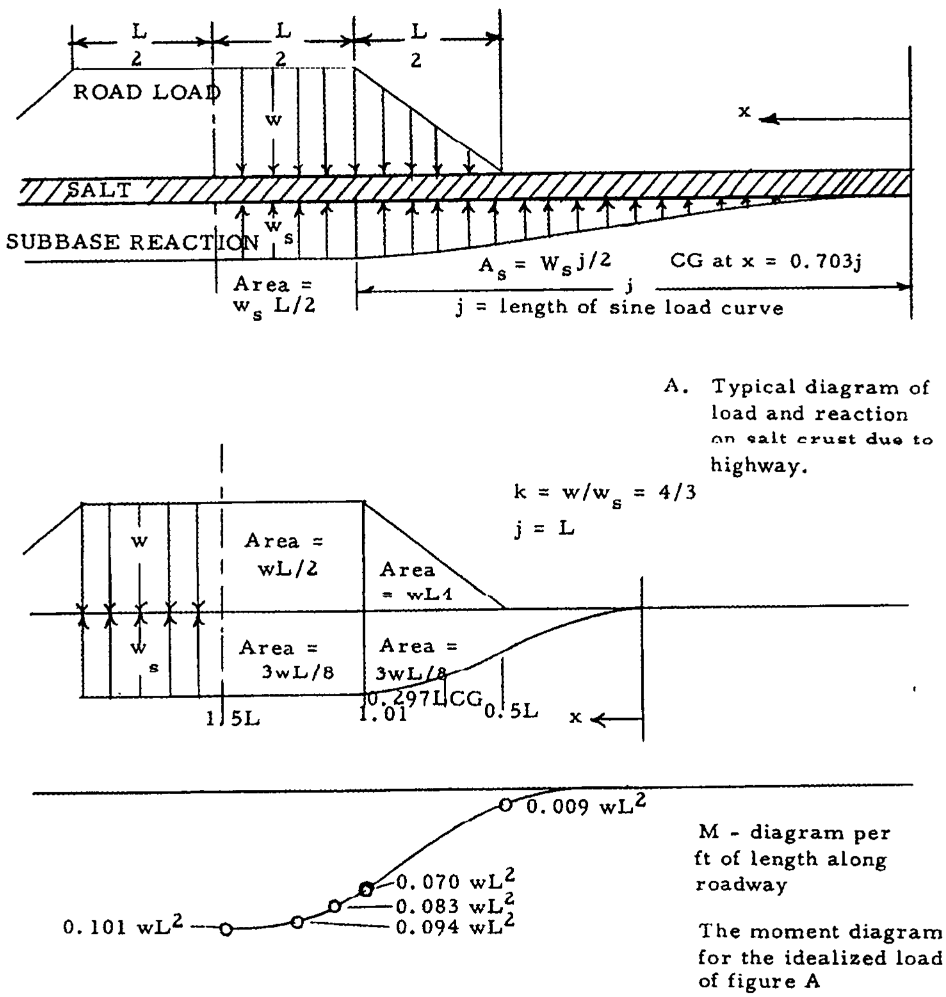


Figure 11-2. Load and moment diagrams for salt crust assuming the vertical road pressure is 4/3 as great as the bearing capacity of the underlying soil.

great as 40 inches because the bottom of the salt layer is so porous as to make its flexural strength undependable. From Fig. 11 of the Report Part V, it is apparent that these stresses far exceed the strength of the salt which ranged from 250 psi at the top to 50 psi at a depth of 12 inches.

Again it must be pointed out that this analysis is based upon the assumption that salt is elastic. Actual tests indicate that this is not the case but that salt creeps in flexure at a stress of less than 18 psi and will, therefore, creep under the road load. The result could only be one of the following:

1. If the ultimate bearing capacity of the clay beneath the salt was less than the load, then the salt layer would simply continue to deform as the road bed settled deeper into the clay. This is improbable since the ultimate bearing capacity of the underlying soil (more than 10 psi) is itself sufficient to carry the road load of about 5 psi.

2. Assuming that the clay consolidates under load it will settle as the salt crust deforms; but the settlement will stop when the bearing capacity of the soil is equal to the load of the highway fill. This is the probable result. As the bearing capacity of the underlying soil increases, the length, j. (Fig. 1) of the sine sub-base reaction curve will decrease and flexural salt stress will decrease to a very small value which the crust can support. We must conclude that the main value of the salt is as fill material to support direct loads only. Flexural strength is negligible.

All of the analysis to this point is invalid if the vertical salt cracks are taken into account. It has been observed that hexagonally or pentagonally shaped blocks of salt are separated by vertical cracks which penetrate the entire thickness of the salt layer. This causes the salt to perform as separate blocks (about 6 feet across) which cannot be depended upon for structural continuity. Again it must be concluded that the salt is of no value except as bulk fill material. It may be argued that salt blocks can transmit shear loads to adjacent blocks. This contributes nothing to the flexural strength of the salt. When an engineer's level was placed on a salt block, it inclined as a man walked around the block, indicating that the salt crust cannot be depended upon to transmit shearing loads. For a short time basis the salt may help support fill material and equipment and may distribute the load during the process of construction and consolidation. On a long-time basis however, the salt can be nothing more than fill material.

A car can travel on a salt crust one inch thick for the following reasons:

1. The car's weight is supported only instantaneously and does not allow sufficient time for the salt to deform by creep nor for the confined soil to consolidate.

2. The tire load causes a three-dimensional rather than a two-dimensional stress pattern and additional support of the tire is developed by the salt to the front and the rear as well as to the sides of the tire. This is not true of the two-dimensional highway stress pattern.

3. It must be recognized that a car is very small in comparison with the road bed, and the loads which might be distributed through the blocks of salt provide a major assist in transferring tire loads to the soil. On the other hand, the blocks of salt are so small in comparison with the size of the roadbed that they may be thought of more in terms of bricks floating in a matrix of mud.

4. Actually the soil beneath the salt has a higher bearing capacity than anticipated. Unconfined compression tests indicate that the bearing capacity of the clay is greater than 10 psi. (Refer to Part VI.) Certainly a strength of 10 psi is adequate to support a highway weighing about 700 pounds per square foot, or about 5 psi. Some unconfined compression tests show bearing capacities less than 10 psi; but in every case the material is not clay, but silt or sand. Loose silt or sand in a saturated state can become quick (liquified) if load is applied instantaneously, but the construction of a highway is not an instantaneous process, and the silt and sand layers will have time to consolidate, and if confined, will certainly carry the load.

Conclusions

1. The salt crust cannot be depended upon to contribute flexural support for the proposed interstate highway. It can serve no better purpose than fill material and a possible temporary means of distributing loads of equipment and highway fill until consolidation of the soil can be accomplished.

2. If the salt must be used as fill, then it is desirable that it be sealed against groundwater flow. A serious limitation of salt is its instability in the presence of groundwater flow and its tendency to dissolve and re-crystaliz thus relieving stresses and reducing the load-carrying capacity. It may be advisable to place a short section of fill on the salt after preliminary soil tests are made. By observing it for a year, the amount of dissolution and recrystalization of the salt may be noted. However, the Western Pacific Railway line has been placed on fill directly on the salt crust. The railroad company's experience and tests on this fill may reveal the necessary information.

Part XII

ANALYSIS OF THE BONNEVILLE SALT CRUST PROBLEM

by

J. E. Christiansen

General Description

The Bonneville salt crust is essentially the salt deposited in the final depression that remained as the ancient Lake Bonneville dried up. This salt island extends approximately 20 miles in a north-south direction and is about 10 miles wide at its greatest width. At the location of the present Highway 40, the salt crust extends from approximately mile post 4 to mile post 12. It extends 10 miles to the north and an equal distance to the south.

The present surface of the salt crust is nearly level except where it has been altered by the operations of the salt companies and Bonneville Ltd. For a number of years it has been utilized for racing, and it is known as the world's fastest race track.

Observations indicate that vertical cracks break the salt crust into slabs somewhat hexagonal in shape. The dimensions of the individual slabs vary, probably with the thickness of the salt crust. In some places there are two or three distinctly different crack patterns superimposed. The major crack pattern, outlined by salt ridges 2 to 3 inches in height, may form slabs about 50 feet across. These slabs are in turn broken into smaller slabs by a similar crack pattern. These slabs are often about 5 feet across varying somewhat in different locations. A third crack pattern is clearly visible in some places. This consists of very fine hairline-type cracks in the surface. It is probable that these fine cracks are not very deep. They are probably caused by temperature stresses at the surface.

Borings by the State Highway Department in 1960 indicate that the salt varies in thickness from less than an inch near the edges to a maximum of across 5 feet. This salt is composed of two distinctly different types: a hard surface crust and a soft layer consisting mainly of a mixture of loose, coarse salt crystals and clay. In the late summer of 1961 the deepened 25-mile ditch completely drained the water from the salt crust within the area surrounded by the ditch. The water table is usually near the surface, except near drains constructed by Bonneville Ltd. These drains lower the water table for a distance of approximately one mile from the drain. During the wetter years, several inches of water is often on the surface during spring months, but by August or September the surface usually dries sufficiently to permit racing.

Structural and Chemical Properties

In 1960 samples of salt were cut from the hard salt crust for structural tests at the USU Engineering Materials Laboratory. These tests were reported in the Report, Part V. At the location where these samples were obtained, a weak horizontal layer existed at a 12-inch depth. Removing salt blocks deeper than 12 inches was impossible because they invariably failed at this weak plane. Structural tests on the salt blocks indicated the following properties:

Flexual strength, surface	250 psi
12-inch depth	50 psi
Modulus of elasticity	1,500 - 10,000 psi
Compressive strength	600 - 1,400 psi
Creep or plastic flow properties	very high

Chemical analyses reported in the Report, Part IV, show that the salt brine has a concentration of about 32 grams per 100 milliliters of solution. This is probably the solubility of the salt crust.

The hard salt crust is very effective in supporting concentrated loads, such as automobile and truck wheel loads. Personal experience indicated that one could safely drive an automobile on a salt crust one inch thick. However, a crust 1/2 inch thick failed. The water table was essentially at the surface: and when the salt crust failed, the automobile wheels settled into the soft mud until the car was resting on the axles.

Effectiveness in Distributing Highway Loads

From the tests and observations mentioned, it might be assumed that salt crust of considerable thickness would be effective in supporting highway fill and dynamic loads. Careful consideration, however, shows that the salt crust would not be effective in distributing highway loads over the underlying soft soil because the salt crust would not act as a continuous plate or beam under the fill nor would it develop flexural stresses, because vertical cracks break it into many relatively small slabs. The load on each slab would essentially be uniformly distributed, supported by a uniform, soft, silty clay.

The slabs would be subjected to compressive stresses equal to the weight of the overlying material. These stresses would be relatively low, approximately 5 psi at the top of the slab. Under the increased loading, the underlying, soft, silty clay will consolidate, and settlement will take place. Tests to date are not sufficient to predict the extent or rate of such settlement; but because of the relatively low loading, it is believed that this settlement will not be excessive. The salt crust, therefore, will act essentially as fill material.

Suitability of Salt as a Fill Material

This reasoning leads to the conclusion that the primary consideration is not the flexural strength of the salt, but rather its suitability as a fill material. To be satisfactory for this purpose, it must have sufficient compressive strength to withstand the loading and it must be stable to the extent that it will not become soluble and be removed from under the fill in solution. There appears to be little question regarding the compressive strength for this purpose. It will not compress or consolidate nearly as much as the underlying soft soil. Therefore, the primary question is: Will the salt tend to dissolve under the conditions to which it will be subjected?

Tests reported in the Report, Part VII, indicate that the soil solution to a considerable depth is essentially a saturated solution. The water pumped from the wells in the pumping tests, Report, Part I and II, is also a saturated solutio This water represents the solution in the blocky clay structure in the depth range of 4 to 9 feet, and in the porous coarse salt zone immediately under the hard salt crust. This salt solution consists of approximately 80 percent sodium chloride, NaCl, or common salt. The remainder is a mixture of calcium, magnesium, and potassium chlorides and sulfates. The solubility of sodium chloride varies relatively little with temperature, from 35.7 percent at 0°C to 39.1 percent at 100°C. This difference in solubility would have little effect on the stability of the salt unless the solution was removed from the salt crust.

Possibly it would be well to review briefly the experiences of the railroad and the highway with respect to the present roadways over the salt crust. Both have experienced considerable trouble, and it is reported that the principal trouble occurs during and immediately following rain storms. This trouble is principally in the form of uneven settlement requiring considerable effort to maintain a smooth, even surface. Evidence indicates that during rainstorms some of the salt dissolves and is removed by water flowing under the roadbed. Solution channels develop, and gravel highway base or railroad ballast falls into these channels, and the surface or rails settle unevenly. Some of the highway trouble evidently is associated with the deep trenches originally excavated along the north side of the highway fill overlying these trenches continues to settle.

The following explanation is believed to be approximately what causes a flow under the roadbed. With the water table very near the surface, it takes only a moderate amount of rain, probably less than an inch, to produce a sheet of water on the surface. Rains are generally accompanied by some wind. The wind blows this water across the salt crust. Where this flow is impeded by any obstruction such as a highway fill, the depth increases to several inches, possibly more than a foot during heavy storms. This creates a considerable hydraulic gradient across the highway and railroad embankments and probably produces considerable flow through the porous salt layer. The water in this instance is only partially saturated and continues to dissolve salt. These flows probably take place largely through cracks in the salt which enlarge into solution channels.

and finally become large enough to undermine the roadway and permit settlement to occur.

If this is the primary reason for the difficulties experienced, would it be possible or feasible to prevent such lateral flow through or under the salt crust? This was attempted, probably with some success, in the construction of the original highway across the salt. Trenches were excavated to the north of the original roadway to provide material for the fill. It is understood that at least one of these trench was refilled with clay to eliminate underflow. That this was not completely effective was very evident during the pumping tests when the water table north of the highway was found to slope toward the deep drain south of the railroad. This effect of the drain extended approximately one mile north of the highway.

It may be both possible and feasible to seal off the flow through the slat layer. Considerable thought must be given to any proposed solution. Cutting trenches parallel with the highway, under or near the shoulder, and backfilling them with clay may be effective but may prove objectionable for other reasons. Such a backfill would have to be placed in a trench filled with water, and it would have to be compacted sufficiently to prevent natural consolidation. Subsequent consolidation of this material would cause settlement of the overlying fill and probably the develop ment of cracks along the shoulder of the highway fill. Evidence indicates that this has occurred over the original trenches along the present Highway 40. The most southerly trench is now under the north edge of the pavement, and cavities below the gravel base have been found. The presence of these trenches is believed to be the cause of part of the difficulty with the present pavement.

If the cutoff trenches were constructed some distance away from the toe of the highway slopes, the salt crust between the trench and the highway fill would have to be sealed to water flow or the trenches would not be effective. Possibly this can be done without excessive cost.

Removal of Salt Crust

Consideration should also be given to the removal of the salt crust and the placement of the fill directly on the underlying silty clay. This may be both difficult to accomplish and very expensive. Under normal conditions the underlying soil would be covered with water when the salt is removed, except where the water table is affected by the Bonneville Drains. This water may be 2 feet or more in depth. Fill material would have to be placed in this water and adequately compacted to a suitable density, or consolidation of the fill may cause excessive and uneven settlement. The actual placement of fill material in a 60-foot wide trench filled with water may present some difficulties. The extra expense of removing the salt and replacing the volume excavated with a fill material would add considerable cost to the project. Before making any recommendation that the salt be removed, actual field experiments should be undertaken to determine the economic feasibility of doing this.

Summary and Conclusions

1. The salt crust would not be effective in distributing the loads over the underlying, soft, silty clay.

2. The salt crust would serve essentially as a fill material.

3. The suitability of the salt crust as a fill material, and not its flexural strength, is of primary consideration.

4. Some of the salt crust may dissolve and be removed from under the highway fill, causing uneven settlement. Rainfall and wind are believed to be the primary cause of this phenomenon.

5. Sealing the salt crust to prevent lateral flow through and under it may be feasible if this can be accomplished satisfactorily and economically without introduc ing other undesirable effects. This suggested solution needs further study.

6. Removal of the salt crust and its replacement with a more suitable material may prove to be the best solution to the problem. This, however, is believed to be both expensive and difficult to accomplish. Before such a solution can be recommended, actual field trials should be undertaken to determine its economic feasibility.

7. Serious consideration should be given to an alternate routing of the highway to the north of the salt crust to avoid the difficulties described herein.

Recommendations

1. Before any specific solution of the salt problem is decided upon, additional studies should be undertaken to determine its practical and economical feasibility.

2. Actual field studies, with standard highway construction equipment, should be undertaken to determine:

a. The feasibility of utilizing the material along the proposed right of way for the highway fill, especially with respect to its placement and compaction.

b. The practicability of constructing cutoff trenches along the edges of the highway fill to prevent lateral flow of water through and under the salt crust.

c. The feasibility of removing the salt crust and replacing it with a suitable fill material.

Part XIII

SUBSURFACE INVESTIGATION

OF U.S. 40-50A, WENDOVER TO KNOLLS

JULY AND AUGUST 1960

by

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SUBSURFACE INVESTIGATION OF US 40-50A, WENDOVER TO KNOLLS
July and August 1960

Purpose of Investigation

The purpose of this investigation was to determine the bearing strength of the fill material and various other physical properties of the material comprising the existing road between Wendover and Knolls, Utah. From the tests run and the observations made it was felt that information could be obtained as to the cause of road failures along US 40-50A, particularly that section which crosses the Bonneville salt crust. The information gained from this investigation could aid in the design and construction of future highways across this section of Utah.

Scope of Investigation

The investigation consisted of two parts undertaken contemporaneously. One part consisted of determining in-place densities and California Bearing Ratios of the existing fill material at various depths down to the original salt crust, or down four feet where the salt crust was absent. Field observations were made of conditions encountered, such as conditions of the road, fill material, and base gravel. Measurements were made of the above highway components and samples were taken for laboratory examination. In addition, all unusual conditions were noted and examined.

The second part, or phase, of the investigation included the obtaining of various samples for laboratory analysis. This included the taking of undisturbed soil samples in 5" diameter Shelby tubes. The Shelby tube samples were taken from the top of the fill material down to the top of the in-place salt crust, where it was present, or to a depth of approximately four feet where the salt crust was absent. Disturbed soil samples were taken of the fill material and base gravel for lab determination of grading, compaction and moisture content, liquid limit, plasticity index, and amount of soluble salts present in the material. Samples were also taken of the asphalt for oil and moisture content determination.

Methods Used for Field Tests and Obtaining Samples

The procedure for obtaining samples and performing the field tests were uniform and consistent for the entire project. The procedure consisted of stripping off the existing asphalt with a backhoe mounted on a Terratrac. The area opened up was approximately 4 to 5 feet wide and ranged in length from 7'± to 9'± (transverse to the direction of the highway) depending on the width of the asphalt. In all cases the test pit was opened up from the edge of the asphalt towards the centerline of the highway and far enough so that the tests could be conducted beneath the outside wheel lane. The test pits were dug on alternating sides of the centerline so that any variation in conditions would be encountered. After the initial opening of the test pit the backhoe was used to excavate the underlying material down to the hard salt crust, or to a depth of 5 to 6 feet ±. This exposed an excellent profile for measuring depths and examining the various levels. At this time samples of the fill material were taken at 6"± intervals, sealed in glass bottles, and retained for later lab determination of moisture content. Temperatures of the clay fill material were taken from the bottom of the base gravel down to a depth of 4 to 5 feet at intervals of one foot ±. The temperature was also taken laterally from the inner edge of the test pit towards the shoulder of the road for a distance of 4 to 5 feet at one foot ± intervals.

After obtaining the moisture content samples and temperatures, the backhoe was again employed to remove the asphalt from a 4' by 4'± area adjacent to the above mentioned pit and along the outside wheel lane. A sample of the asphalt was taken at each test area sealed and retained in a metal can for oil and moisture content determination and for sieve analysis.

Samples were also taken of the base gravel at most of the test pits, and the clay fill material at each of the test pits. The base gravel was tested in the laboratory for grading, liquid limit and plastic index by standard A.A.S.H.O. testing procedures. The clay fill material was tested in the lab for grading, liquid limit and plastic index, soluble salts, and compaction. The compaction was accomplished by the T-180 Method D (modified proctor) procedure, to determine dry densities and optimum moistures. The other tests were run according to standard A.A.S.H.O. Specifications.

Field C.B.R. (California Bearing Ratio) tests were conducted at the top of the clay fill material by a soiltest field C.B.R. unit using the weight of the Terratrac as the reaction force for the penetration rod. C.B.R. tests were conducted at 1'± vertical intervals from the top of the clay to a depth of approximately 4' or to approximately 1' above the hard salt crust wherever it was present. In all cases care was taken to scrape away all of the base gravel imbedded in the top of the clay. Three separate sets of readings were taken at the top elevation and one set each at the succeeding lower elevations. For each set of C.B.R. readings, moisture samples were taken for analysis by the District #2 lab. Also at each elevation, in-place densities were determined with a Washington Dense-O-Meter, using a mixture of water and soluble oil in a rubber membrane. Representative samples from the material removed for density determinations were retained in glass bottles for moisture content determination.

Undisturbed soil samples were obtained for the Central Testing Laboratory by forcing 5" diameter, 6" long, relatively thin walled Shelby tubes down into the fill material. The Shelby tubes were forced down by hammering with a ten pound sledge hammer. This provided a nearly continuous undisturbed sample in 6"± sections from the top of the fill material to a depth of approximately 4'. Recovery in the tubes ranged from 90% to 100%. The Shelby tubes were sealed at both ends by the use of high-melting-point wax.

Lab Analysis Procedure

Mechanical analysis, liquid limits, plastic indexes, oil content, and moisture contents were all determined according to standard A.A.S.H.O. testing procedures. Soluble salt content was determined by standard procedures used by the Utah State Highway Departments Central Materials Laboratory. Dry densities and optimum moistures were obtained by compacting the material by the T-180 Method D (modified proctor) procedure. Modified proctor tests were conducted with both regular culinary water and saline water obtained from the salt crust adjacent to the highway.

Summarized Test Results

The detailed results of the tests conducted on the base gravel, clay fill material, and asphalt are tabulated on separate sheets. Following is a generalized summary of the results.

The revised A.A.S.H.O. classification of the base gravel is A-1-a(0) for all of the samples taken with the exception of those taken at mile posts 3 and 35, both of which were A-1-b(0). The liquid limits ranged from 16.2 to 18.0 with the exception of mile posts 35 and 40 which were 24.0 and 21.9 respectively. All of the base gravel samples except those from mile posts 35 and 40 were non-plastic.

The base gravel samples taken from mile posts 9 and 30 meet specifications for base gravel, all of the other samples contain an excess of -#200 sieve size material.

Four of the asphalt samples did not meet grading specifications for type A plant mix bituminous material. The samples not meeting specifications were taken from mile posts 13, 15, 35 and 40. The samples from mile posts 13 and 40 contained an excess of -#200 material while the one from mile post 15 showed a small deficiency of -#200 material. The sample from mile post 35 had a deficiency of -#4 material.

The oil content ranged from 4.08% to 6.40%. The moisture content ranged from 0.38% to 1.28% from mile posts 3 through 35. The moisture content of the sample taken from mile post 40 was an abnormal 3.24%. It was also noted that this section of highway contained an excess amount of asphalt on the road surface.

The road fill material was sampled and tested at each test pit. This material was fairly homogeneous clay throughout except at mile post 40 where it had an appreciable amount of sand. In many of the test pits not underlain by the hard salt crust, the contact between the fill material and the in-place material was difficult to determine due to the homogeneity of the two materials.

The A.A.S.H.O. classification of the fill material ranged between an A-4(4) and a A-7-6(11) with the greater majority of the samples falling in the A-6 class with group indexes of 8, 9, or 10.

The liquid limit range was from 46.1 to 27.8. The plastic indexes varied from 17.5 to 5.0 with the sandy sample from mile post 40 being non-plastic.

Field C.B.R. values obtained with the soiltest field C.B.R. unit are tabulated at the end of this report. In general, only a small number of the corrected values were above 25% of standard. The majority of the values were in the 5% to 15% of standard range. These values generally decreased with depth of penetration and also with depth at which the tests were conducted, while the moisture content of the fill material increased with depth.

The results of the in-place density and compaction tests show that the fill material directly below the base gravel has a compacted percentage usually between 85 and 90, notable exceptions being at mile post #5 (1.0' depth below the surface and 14.4 percent moisture) which had 103.1 percent compaction and mile post #35 (1.4' depth below the surface and 46.0 percent moisture) which had 64.8 percent compaction. The percent of moisture in the fill material increases with depth except at mile post #6, #11, #30 and #35 where it seems to fluctuate with depth. The percent moisture at mile post #30 (3.1' depth below the road surface) is 64.5. This is an unusually high water content and is reflected in the low dry density of 59.7 lb./cu. ft. found at this mile post.

The temperature of the fill material below 2'± decreased with depth as a straight line function. The temperature of the fill material above 2'± seemed to vary proportionately as the temperature of the surface and air varied.

Field Observations

Observations of the road surface condition were made at each mile post where test pits were dug. In general the condition of the road was good to fair. At mile post 7 the road appeared to be in poor condition. Between mile posts 5 and 12 the road generally exhibited some transverse and longitudinal cracks and a few depressions and swells.

At mile post 9 the underlying clay and salt crust showed a definite profile sloping towards the right, or south shoulder of the road. This sloping and the resulting depressions in the road surface are probably due to the leaching out of the salt at this section of the road. At other sections where test pits were dug and surface depressions were noted, the leaching effects were not in evidence. It was therefore assumed that the major cause of the surface depressions was a lack of compaction of the fill material during construction rather than the leaching out of the salt.

At mile post 3 some unusual solution channels were noted as the test pit was dug. These channels were approximately 1/2" to 3/4" in diameter and did not seem to have any preferred orientation. Some were nearly vertical, some nearly horizontal, and so far as could be determined, all of the channels were interconnected.

At approximately mile post 8 a tandem roller broke through the asphalt while rolling a new asphalt surface during resurfacing of US 40-50A. When this section was opened up with a backhoe it could be seen that the base gravel had fallen completely away from beneath the asphalt near the edge of the road, leaving only scattered "bridges" of compacted, in-place base gravel which was carrying the load of the outer two foot ± edge of asphalt. Further excavation confirmed the fact that the outer edge (north side of highway) of the asphalt is directly over the old cut off trench excavated during the initial construction of US 40-50A. This trench was backfilled with loose, uncompacted clay for the purpose of preventing ground water from circulating under the newly constructed highway. Since then the present road has been built out over the filled trench. The impact of traffic along the road has caused the material in the trench to gradually settle, resulting in failure of the road. Free water was reached at approximately 7 feet in depth in this trench.

Conclusion

In conclusion it should be noted that the primary cause of the road failure must be attributed to the unconsolidated, uncompacted underlying highway embankment. This embankment was placed at a time when knowledge about compaction was in its infancy.

The clay fill material since its initial placement has gradually settled and received some compaction from the increasing, heavier highway traffic. The ultimate outcome has resulted in surface failure comprised of cracks, depressions and rolls.

Although no conclusive evidence was found to substantiate the theory that fresh water has been leaching the salt from beneath the highway embankment, we must adhere to the fact that this theory is feasible and has probably caused some failures. The observations made and the tests conducted show that the main cause of failures is directly connected to the lack of compaction in the initial placing of the embankment materials.

It is recommended that extreme caution be exercised in compacting the embankment for the new Interstate Highway between Wendover and Knolls. If side borrow or material from trench excavations is used, the excess moisture must be eliminated to insure the obtaining of the optimum moisture content and the maximum dry density.

FIELD AND LABORATORY DRY DENSITY DATA

M.P.	Depth (Ft.)	Field: Dry Density (lb/cu ft)	Field: Moisture (%)	Proctor: Dry Density (lb/cu ft)	Proctor: Optimum Moisture	Compaction (%)
3	2.0	99.4	27.0	108.5	19.8	91.6
	3.5	83.8	36.4	108.5	19.8	77.2
4	1.4	101.0	26.0	112.8	17.3	89.6
	2.8	94.1	28.7	112.8	17.3	83.4
5	1.0	123.2	14.4	119.5	15.1	103.1
	2.4	102.7	23.4	119.5	15.1	85.8
6	1.7	98.5	24.2	112.4	17.0	87.6
	2.7	103.6	17.0	112.4	17.0	92.2
7	2.1	93.2	30.1	110.5	17.9	84.3
8	1.8	98.3	24.1	113.4	17.2	86.6
	3.1	79.0	29.1	113.4	17.2	69.6
9	1.8	100.6	26.2	112.8	17.2	89.5
	2.6	97.4	30.5	112.8	17.2	86.3
10	1.8	92.5	31.4	111.3	17.9	83.0
	2.8	87.3	33.6	111.3	17.9	78.4
11	1.5	101.5	25.1	112.8	17.7	90.0
	3.1	106.7	16.4	112.8	17.7	94.5
12	1.95	96.2	26.9	113.5	16.6	84.7
	2.9	100.1	22.4	113.5	16.6	88.1
13	1.5	94.7	29.4	113.0	17.6	83.8
	3.25	87.0	32.3	113.0	17.6	77.0
15	1.4	104.4	22.6	117.5	15.2	88.0
	2.3	94.7	30.5	117.5	15.2	80.6
20	1.4	105.1	22.6	119.6	15.9	88.0
25	1.8	95.2	28.3	112.6	17.0	84.5
	2.8	91.1	29.7	112.6	17.0	80.9
	3.6	80.9	40.7	112.6	17.0	71.8
30	2.2	92.6	36.0	104.0	21.1	89.0
	3.1	62.1	64.5	104.0	21.1	59.7
	4.3	71.5	46.9	104.0	21.1	68.8
35	1.4	71.6	46.0	110.5	20.0	64.8
	2.5	77.4	36.7	110.5	20.0	70.0
	4.2	75.9	44.5	110.5	20.0	68.7
40	1.5	91.8	18.9	112.3	17.6	81.7
	2.6	89.1	21.8	112.3	17.6	79.3

Plate 1

CALIFORNIA BEARING RATIO VALUES

(Average values of corrected percentages of standard)

Sheet 1 of 2 Sheets

		FIELD VALUES							LAB VALUES					%Moist.		
		Penetration (In.)							Penetration (In.)					Top	Swell	D.D.
M.P.	Depth	0.1	0.2	0.3	0.4	0.5	%Moist	D.D.	0.1	0.2	0.3	0.4	0.5	Inch	%	Lb/ft^3
3	1.9'	17.7	18.6	18.7	18.2	18.4	26.3	99.4	7.5	9.9	11	12	14	22.5	1.6	112.0
	3.4'	10.6	8.7	7.7	7.0	6.6	34.4	83.8	*8.2	12	14	15	16	28.3	3.2	107.7
4	1.4'	19.5	19.1	17.6	16.2	15.4	24.6	101.0								
	2.8'	6.3	5.1	4.4	3.9	3.7	28.8	94.1								
5	1.0'	16.6	20.5	22.2	21.8	21.2	14.5	123.2								
	2.1'	4.2	3.9	3.6	3.4	3.1	23.2	102.7								
6	1.7'	10.5	10.8	10.6	10.2	10.0	22.1	98.5	11	15	18	20	23	20.3	2.6	114.0
	2.7'	5.1	4.6	4.3	3.8	3.6	29.6	103.6								
7	2.1'	5.5	6.2	6.0	5.5	5.2	27.0	93.2	4.8	6.5	7.9	9.0	10.2	25.7	3.13	113.2
8	1.7'	4.2	5.0	5.5	5.7	5.6	23.4	98.3								
	3.1'	2.0	2.1	2.0	1.9	1.7	30.2	79.0								
9	1.8'	12.9	12.4	11.4	10.4	9.9	24.7	100.6	6.5	8.7	10	12	14	18.8	3.1	116.0
	2.6'	4.6	4.0	3.6	3.3	3.1	25.5	97.4								
10	1.8'	8.2	7.2	6.2	5.4	5.0	27.7	92.5								
	2.8'	2.1	2.1	2.0	1.8	1.8	26.3	87.3								
11	1.5'	12.9	13.0	12.2	11.2	10.6	21.9	101.5								
	3.1'	2.8	3.1	3.1	2.8	2.8	14.9	106.7								
12	1.95'	7.9	7.5	7.2	6.4	6.0	23.7	96.2	8.5	12	15	18	20	20.0	2.8	112.0
	2.9'	3.0	3.1	2.9	2.5	2.3	16.1	100.1	*5.0	7.2	9.1	10.9	12.6	24.2	3.5	115.8

CALIFORNIA BEARING RATIO VALUES

(Average values of corrected percentages of standard)

Sheet 2 of 2 Sheets

		FIELD VALUES							LAB VALUES					%Moist.		
		Penetration (In.)							Penetration (In.)					Top	Swell	D.D.
M.P.	Depth	0.1	0.2	0.3	0.4	0.5	%Moist.	D.D.	0.1	0.2	0.3	0.4	0.5	Inch	%	Lb/ft^3
13	1.8'	15.6	15.4	14.1	12.8	12.3	25.4	94.7								
	3.3'	1.1	1.1	0.9	0.9	0.8	34.8	87.0								
15	1.35'	16.5	15.1	13.5	12.0	11.2	22.1	104.4								
	2.3'	3.2	3.1	3.0	2.8	2.7	31.4	94.7								
	3.0'	2.1	1.9	1.7	1.7	1.7	23.8									
20	1.5'	21.4	21.4	19.6	17.9	17.1	21.5	105.1	15	18	21	22	23	19.1	2.1	119.6
	2.8'	26.5	24.5	21.2	18.6	17.2	13.9									
25	1.8'	12.0	11.6	10.8	9.7	9.3	24.9	95.2								
	2.8'	8.1	7.3	6.8	6.3	6.1	28.5	91.1								
	3.6'	3.0	2.7	2.5	2.2	2.1	32.3	80.9								
30	2.0'	14.4	13.1	11.9	10.7	10.0	30.7	92.6	6.7	7.4	8.2	8.2	8.5	26.3	2.0	101.0
	3.1'	5.8	4.9	4.4	4.1	4.0	28.7	62.1								
	4.3'	2.0	1.9	1.6	1.6	1.5	48.0	71.5								
35	1.4'	6.2	5.8	5.2	4.8	4.7	52.0	71.6								
	2.4'	2.2	2.3	2.2	2.0	1.9	34.2	77.4								
	4.2'	3.2	2.1	2.1	1.9	1.8	46.9	75.9								
40	1.4'	29.9	27.2	22.8	19.9	18.4	16.9	91.8	83	82	73	67	67	19.2	0.5	109.0
	2.7'	7.9	8.3	8.2	8.0	8.2	21.0	89.1	*65	75	73	70	64	23.6	0.1	110.0

*= C.B.R. Laboratory Re-Run Results

Plate -2

ROAD FILL MATERIAL SAMPLE DATA

M.P.	Depth		Sieve Analysis (% Passing)						LL	PI	T-180 Method D (Modified Proctor)				AASHO Classi-fication	Soluble Salt Content %
											Culinary Water		Saline Water			
	From	To	1"	3/4"	#4	#10	#40	#200			Opt. Moist.	DD	Opt. Moist.	DD		
3	1.9	4.5	100	100	99.8	99.6	98.8	92.5	46.1	17.5	19.8	108.5	18.8	111.5	A-7-5(12)	6.4
4	1.5	4.0	100	99.9	98.9	97.1	95.2	86.7	36.7	13.6	17.3	112.8			A-6(9)	10.8
5	1.0	3.0	100	100	99.9	99.1	96.4	78.4	30.8	8.7	15.1	119.5			A-4(8)	12.0
6	1.4	3.4	99.6	99.5	95.6	92.2	89.5	84.6	34.5	12.3	17.0	112.4			A-6(9)	13.2
7	2.0	3.3	100	100	98.8	97.2	95.4	87.9	36.8	5.0	17.9	110.5			A-4(8)	7.2
8	1.7	4.0	100	99.9	98.0	94.8	90.9	79.0	36.6	13.4	17.2	113.4			A-6(10)	11.8
9	1.5	3.8	100	100	98.3	97.3	96.4	87.8	35.7	13.8	17.2	112.8			A-6(10)	12.0
10	1.8	3.8	100	99.8	98.7	96.9	95.0	86.5	37.0	12.9	17.9	111.3			A-6(9)	8.0
11	1.4	4.25	100	100	98.7	97.8	96.6	90.1	36.3	12.8	17.7	112.8			A-6(9)	17.2
12	1.9	4.1	100	100	98.7	94.3	90.5	77.1	31.9	10.2	16.6	113.5			A-6(8)	16.8
13	1.3	3.4	100	99.8	99.3	98.0	96.8	86.9	37.5	14.0	17.6	113.0			A-6(10)	8.8
15	1.3	5.0	100	99.9	97.4	94.1	89.8	69.8	28.8	10.3	15.2	117.5	14.5	117.4	A-6(8)	7.6
20	1.5	5.0	100	99.5	97.8	96.2	91.4	61.2	27.8	9.3	15.9	119.6			A-4(5)	16.0
25	1.6	4.5	100	100	99.9	99.6	98.7	90.9	36.8	14.1	17.0	112.6			A-6(10)	10.4
30	2.0	5.0	100	99.8	99.2	97.2	86.0	70.9	43.3	15.8	21.1	104.0			A-7-6(11)	10.8
35	1.2	3.5	100	100	99.9	97.8	96.2	90.6	40.5	16.0	20.0	110.5	17.2	113.6	A-7-5(10)	20.0
40	1.1	4.7	100	100	99.9	99.7	89.1	55.0	23.2	N.P.	17.6	112.3			A-4(4)	23.2

Plate-3

BASE GRAVEL

M.P.	Depth (Ft.) From	To	Sieve Analysis (%Passing) 1"	3/4"	3/8"	#4	#10	#40	#200	LL	PI	AASHO Classification
3	0.6'	1.9'	99.1	97.2	84.4	70.6	52.4	33.3	15.9	16.7	NP	A-1-b(0)
4	0.6'	1.5'	No Sample									
5	0.5'	1.0'	94.6	91.7	74.1	52.5	33.2	22.9	13.3	16.4	NP	A-1-a(0)
6	0.7'	1.4'	98.7	96.6	77.1	57.1	41.7	28.5	14.4	16.2	NP	A-1-a(0)
7	0.75'	2.0'	99.3	97.7	82.4	61.9	40.2	23.8	11.0	16.2	NP	A-1-a(0)
8	0.8'	1.7'	No Sample									
9	0.7'	1.5'	99.0	95.4	71.2	49.1	30.8	20.3	9.9	17.2	NP	A-1-a(0)
10	0.9'	1.8'	No Sample									
11	0.45'	1.4'	No Sample									
12	0.5'	1.9'	No Sample									
13	0.7'	1.3'	98.9	97.0	79.1	55.0	36.1	21.7	10.5	17.6	NP	A-1-a(0)
15	0.4'	1.3'	No Sample									
20	0.35'	1.5'	99.9	96.6	77.4	57.0	39.7	24.4	10.2	18.0	NP	A-1-a(0)
25	0.6'	1.6'	99.8	97.8	78.5	54.3	36.0	24.3	14.6	17.8	NP	A-1-a(0)

BASE GRAVEL

	Depth (Ft.)		Sieve Analysis (%Passing)									AASHO Classifi-
M.P.	From	To	1"	3/4"	3/8"	#4	#10	#40	#200	LL	PI	cation
30	0.7'	2.0'	100	97.9	72.0	49.9	34.7	22.8	9.3	17.0	NP	A-1-a(0)
35	0.5'	1.2'	96.2	92.6	70.5	50.1	34.2	23.7	17.5	24.0	5.1	A-1-b(0)
40	0.4'	1.1'	82.3	74.9	59.9	48.8	38.7	26.3	14.3	21.9	3.5	A-1-a(0)

ASPHALT SAMPLE DATA

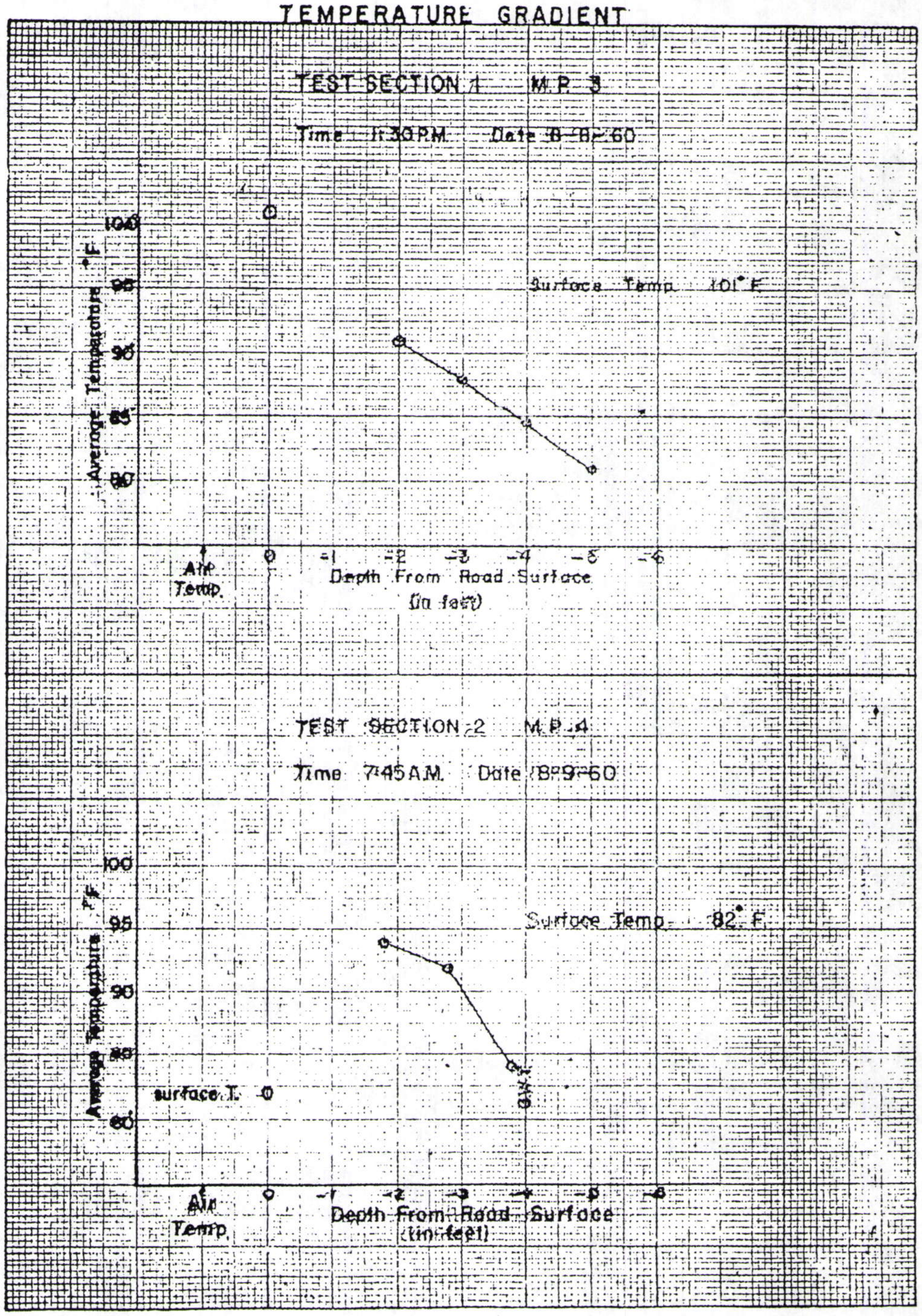
M.P.	Depth (Ft.)		Asphalt Content (%)	Moisture Content (%)	Sieve Analysis (% Passing)						
	From	To			1"	3/4"	3/8"	#4	#10	#40	#200
3	0.0'	0.6'	4.30	0.56	100	96.1	80.0	60.2	38.6	21.9	7.5
4	0.0'	0.6'	6.22	0.44	100	100	83.7	62.4	39.5	22.3	7.2
5	0.0'	0.5'	4.12	1.20	100	100	85.5	58.9	37.4	20.9	6.7
6	0.0'	0.7'	4.08	0.62	100	100	69.5	48.2	34.1	23.3	9.1
7	0.0'	0.75'	5.60	1.08	100	96.9	79.2	60.3	39.8	23.9	10.0
8	0.0'	0.8'	5.94	0.70	100	100	83.3	58.6	39.3	24.4	8.8
9	0.0'	0.7'	4.46	0.38	100	100	78.3	56.4	37.7	23.8	8.4
10	0.0'	0.9'	5.40	0.66	100	100	82.7	60.6	39.6	24.8	9.0
11	0.0'	0.45'	5.16	0.88	100	98.5	77.9	56.3	33.9	19.1	7.5
12	0.0'	0.5'	5.80	0.64	100	91.9	71.3	53.5	33.1	18.1	6.8
13	0.0'	0.7'	4.72	0.72	100	96.6	69.2	48.3	30.8	19.6	10.6
15	0.0'	0.4'	6.46	0.84	100	97.1	75.0	54.3	34.1	19.3	4.8
20	0.0'	0.35'	4.38	1.28	100	97.4	72.3	52.0	32.6	18.5	6.1
25	0.0'	0.6'	5.14	1.08	100	100	79.0	58.9	40.8	25.5	8.3
30	0.0'	0.7'	5.06	0.94	100	93.4	73.5	52.5	34.3	21.5	8.6
35	0.0'	0.5'	4.72	1.28	100	92.4	57.6	43.6	31.9	22.7	8.6
40	0.0'	0.4'	6.04	3.24	100	93.2	62.5	48.3	35.6	24.9	13.6

M.P. abbreviation for mile post (zero mile post is at the Utah-Nevada state line)

Plate 5

-12-

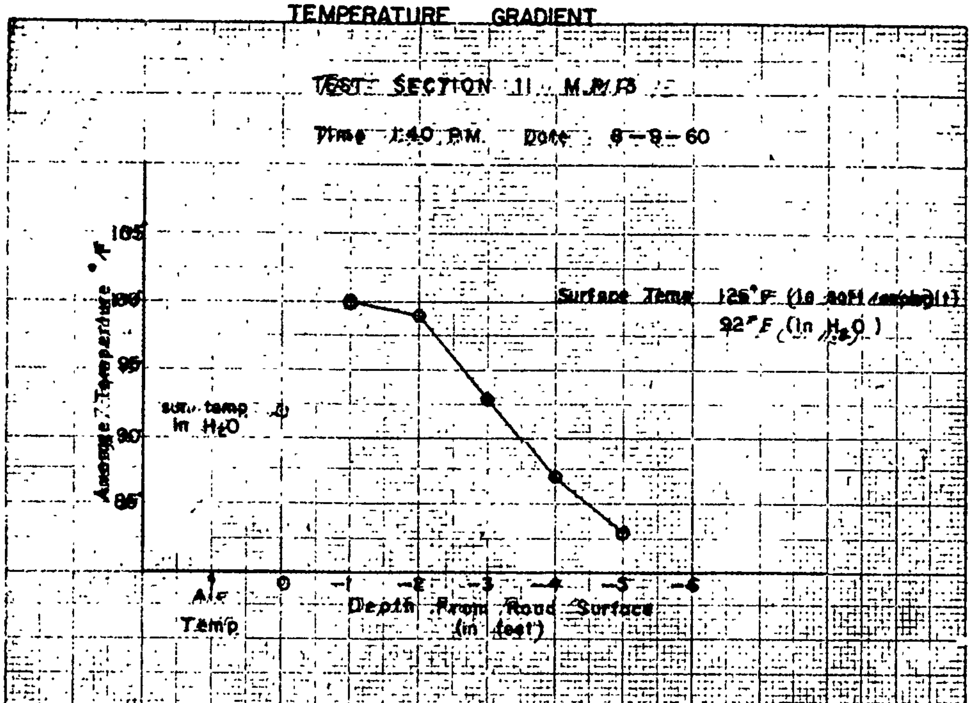
TEMPERATURE GRADIENT

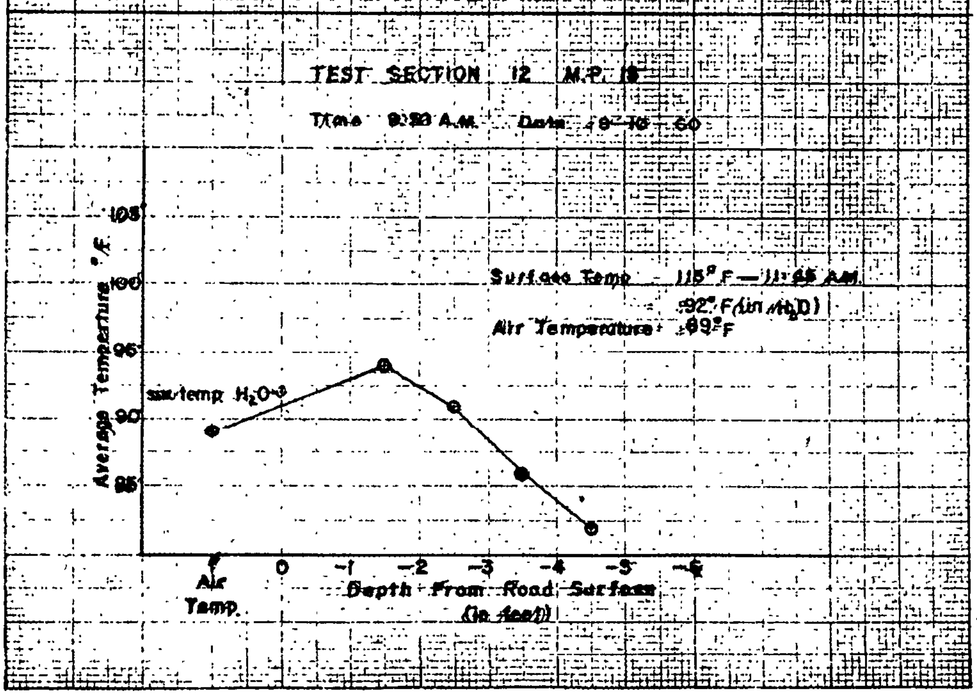


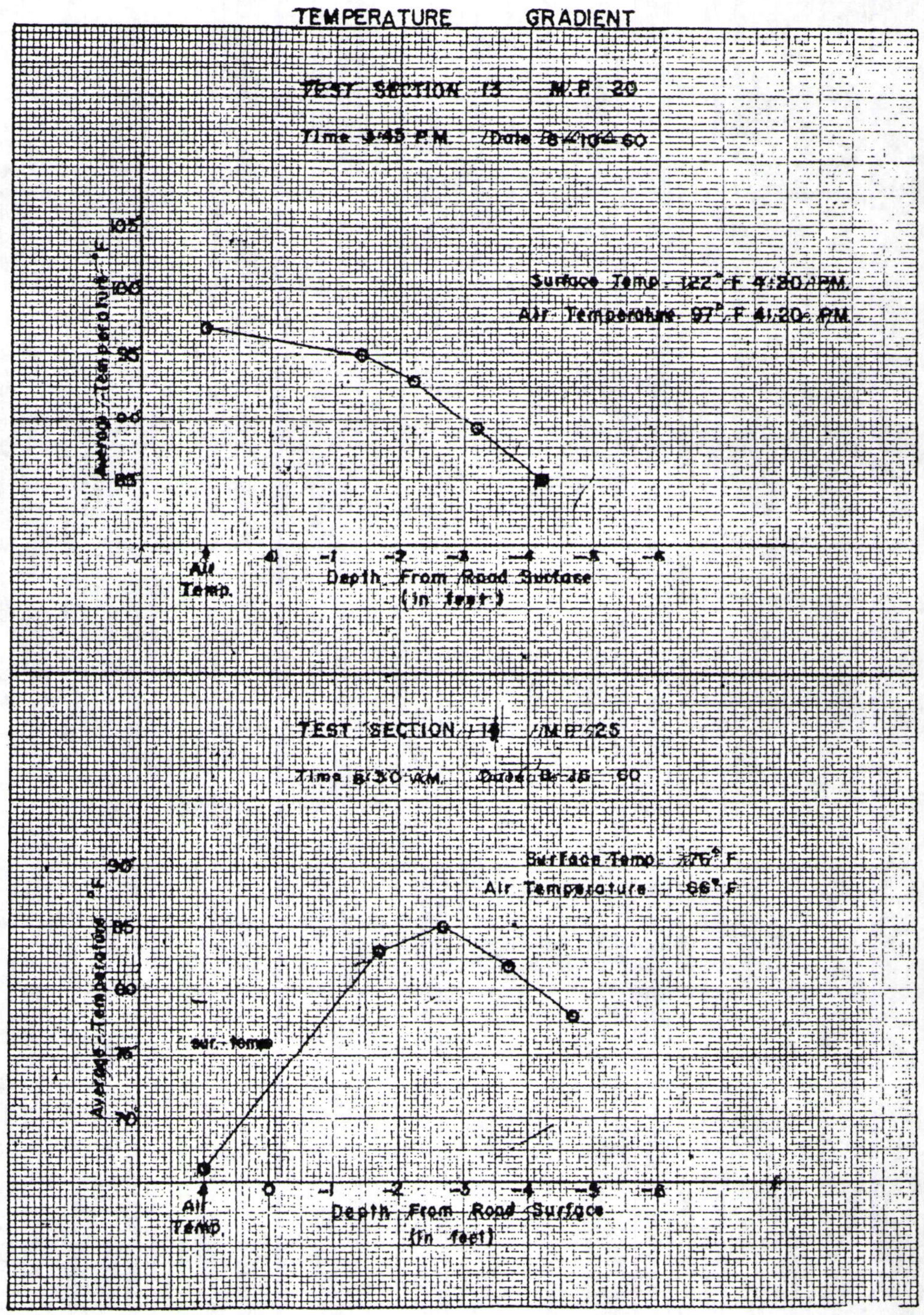
Form No. 999—20 Squares to Inch

-13-

TEMPERATURE GRADIENT



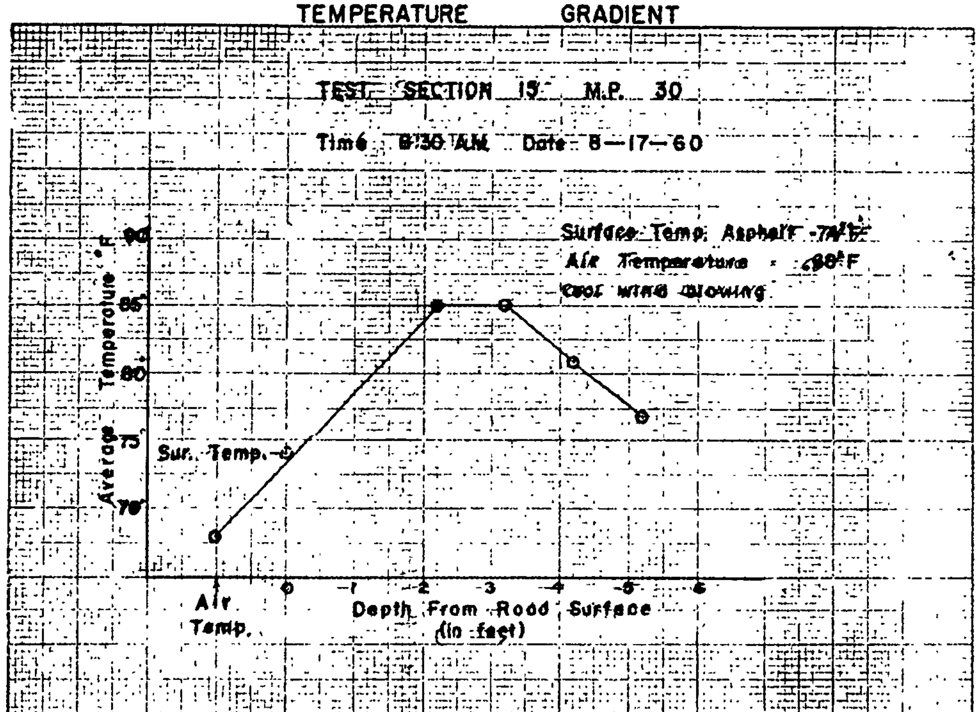


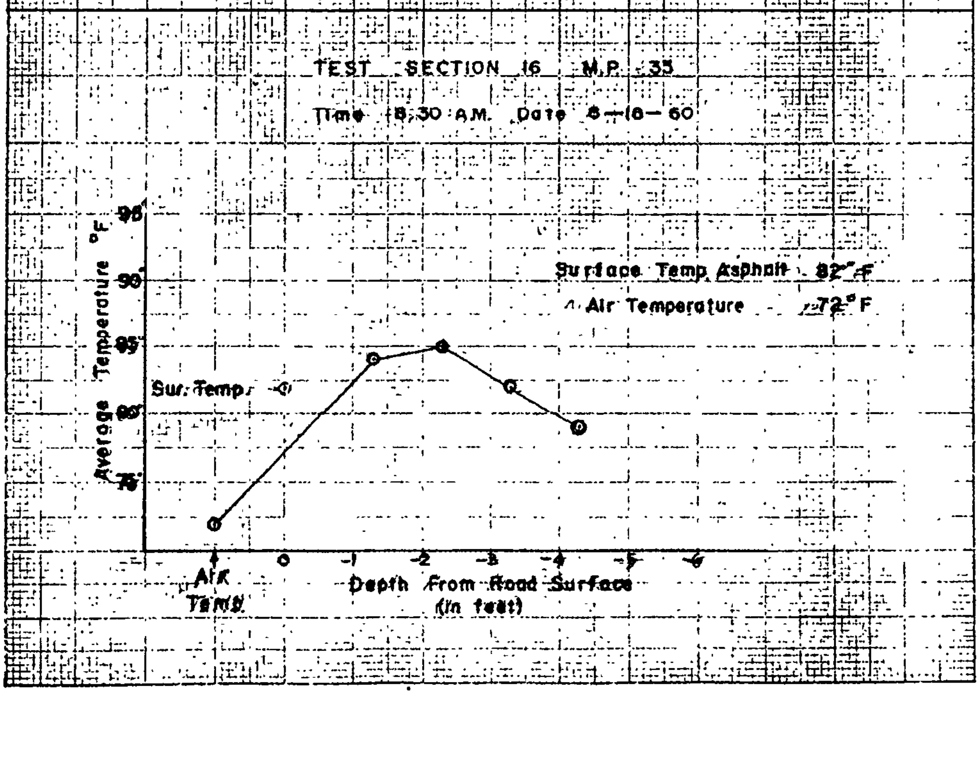


Form No. 999-20 Squares to Inch

-15-

TEMPERATURE GRADIENT

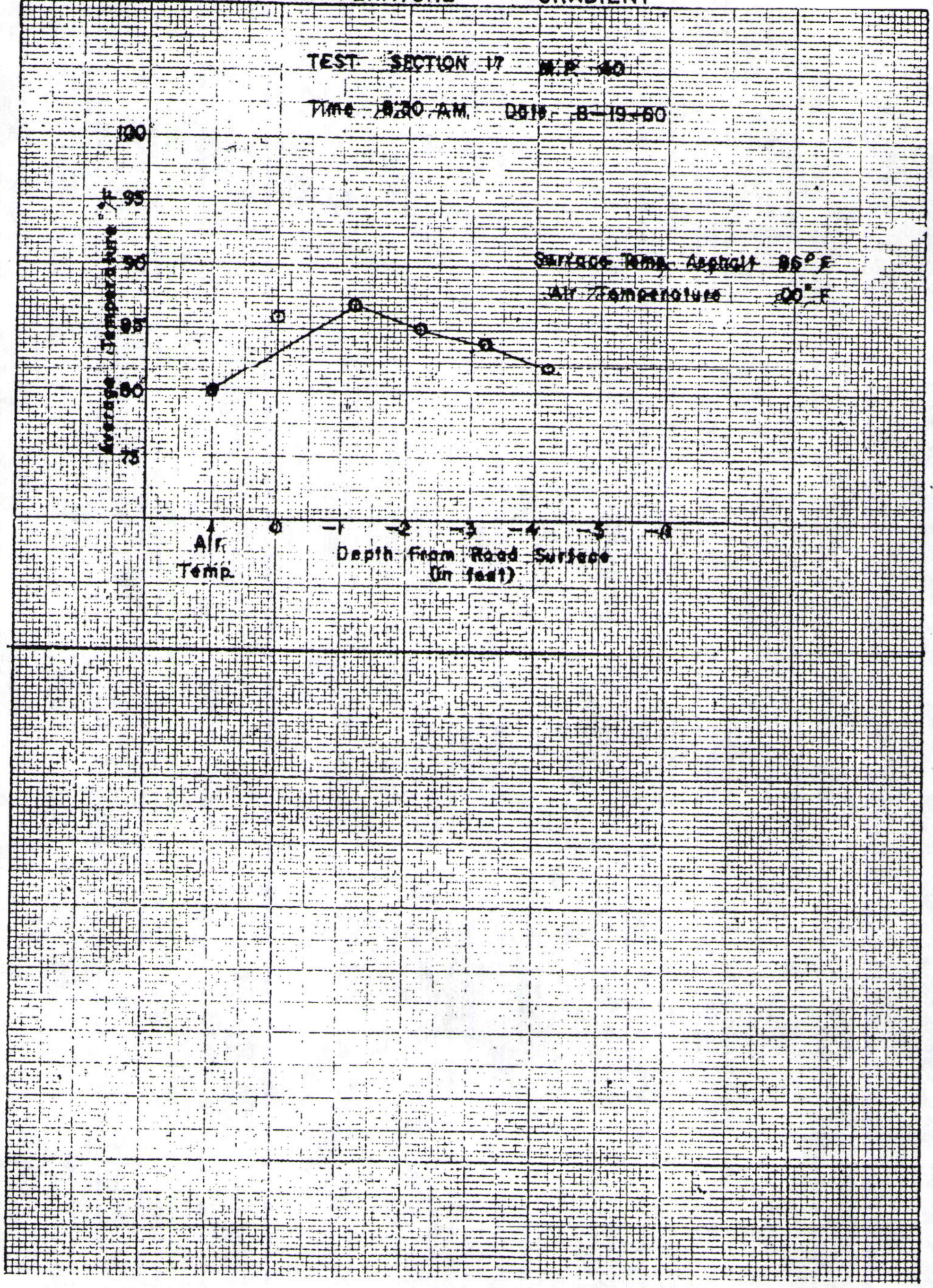




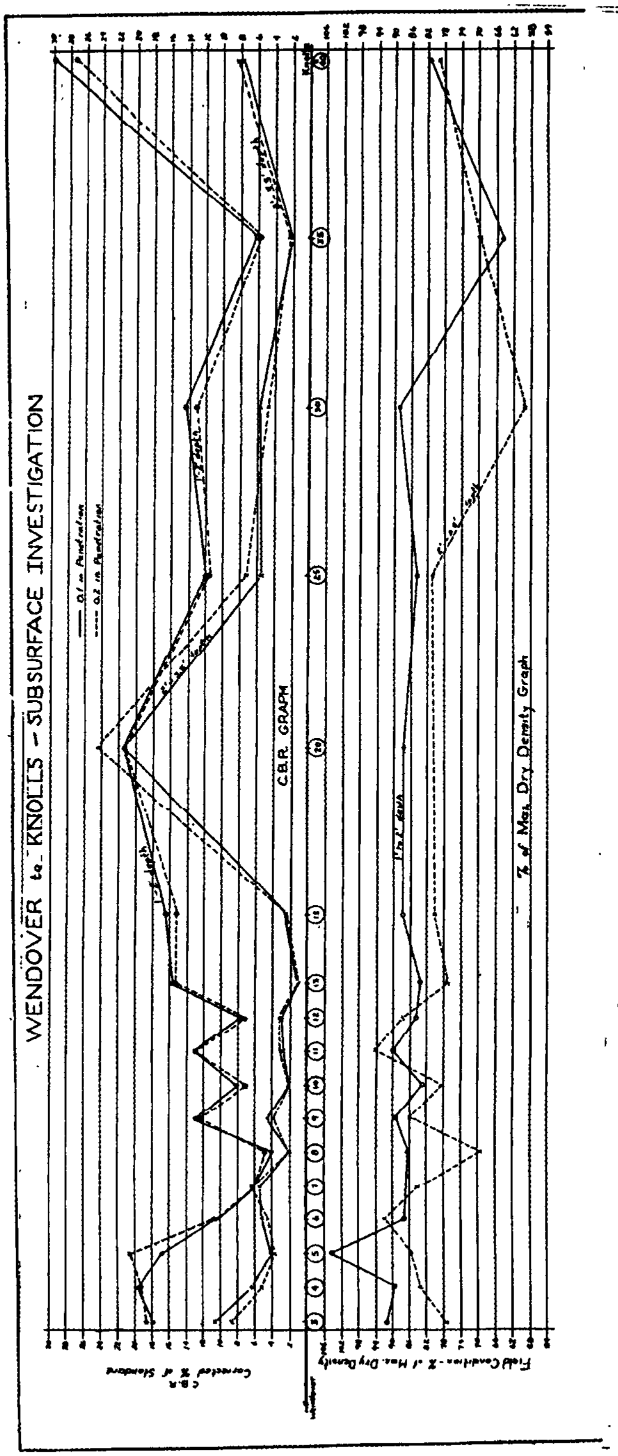
Form No. 999—20 Squares to Inch

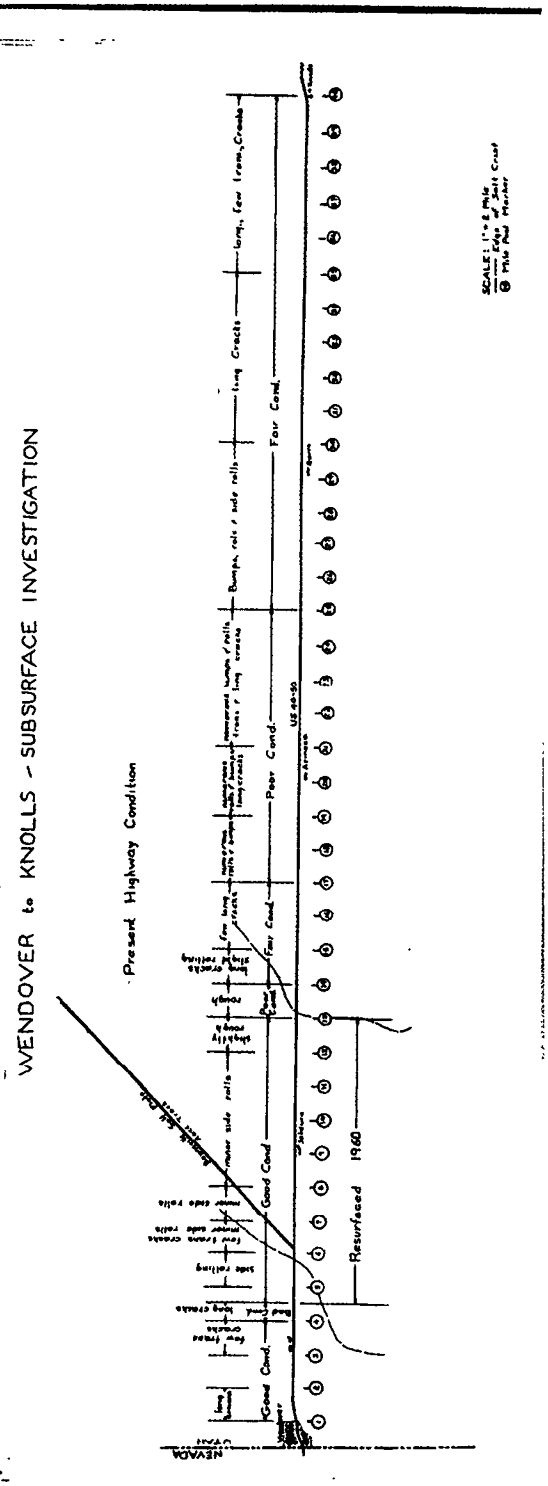
-16-

TEMPERATURE GRADIENT



Form No. 999-20 Squares to Inch


WENDOVER to KNOLLS - SUBSURFACE INVESTIGATION
C.B.R. GRAPH
% of Max. Dry Density Graph


WENDOVER to KNOLLS - SUBSURFACE INVESTIGATION
Present Highway Condition
Resurfaced 1960
Good Cond.
Fair Cond.
Poor Cond.